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Uncertainties in redesigning an existing quay wall

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Abstract. A feasibility study was carried out on the redesigning of an existing quay wall of a core harbor in the Netherlands, which was more than 45 years old. The geotechnical behavior of the existing quay wall, especially because of its age, as a response to the maximum load change is undoubtedly the uncertain parameter here. The design and redesign aspects have been considered: the evaluation of the current situation, the remaining lifetime of the structure and other aspects like corrosion and fatigue of the construction materials and the change in design standards between the past and the future situations. No monitoring and measurements were available. Inspection on steel structural elements showed some corrosion, to which extend was unknown. The history of load usage of the quay wall was not registered. After a preliminary redesign, it is clear that some uncertainties would remain. It was concluded that the lack of information in the current situation constitutes the main obstacle to a straightforward redesign and the use of Finite Element Method modelling reveals a failure mechanism, which was not encountered earlier. Because of the technical risks, a redesign of such existing and complex quay wall would necessitate an extensive design procedure to increase reliability.

Keywords. Limit state design, structures, quay walls, risks, reliability, geotechnical uncertainties.

1. Introduction

Merchant shipping is the lifeblood of the world economy. Since the last decade, a transformation is going on in shipping transport. Ships are bigger in size and volume. Nowadays, a much bigger volume of goods can be transported by one ship (De Gijt, 2010). Therefore harbor infrastructures need to be adapted in order to cope with this new situation: larger ships with generally heavier shipment to be transferred from sea to the hinterland. Nowadays, the size of dry cargo ships or bulk carriers varies between 200 and 400 m. These ships can have 2-4.10^5 dead weight tonnage (DWT).

In the ore harbor in the Netherlands, near IJmuiden, a transformation of the existing quay walls was necessary to enable a growth in the production of steel, see Figure 1. The owner of the quay walls commissioned a study to check the feasibility of an eventual transformation or upgrading of the existing quay walls.

Figure 1. Airborne view of the ore harbor

The main requirements of the rebuilding here are:

- Deepening of the bottom of the waterways from -18 m to -20 m according to the Amsterdam Ordnance Datum (NAP) which comes to 2 m deepening.
- An increase of the load on the quay surface from a bulk density of 22 to 31 kN/m^3 and an increase of the height of the iron core depository behind the quay from 16 m to 17 m.

All these mean a maximum effective increase of 45 % of the total vertical load on the quay wall structure. This would surely lead to additional horizontal forces behind the retention wall and to additional tension forces on the existing piles and anchors if no reinforcement is undertaken.

The application of the new conditions means that the quay walls have to retain more loads while getting less support of the soil. In geotechnical terms, it means larger active earth pressure and lower passive earth pressure.

Besides the pure geotechnical challenges of such an upgrade, the above mentioned requirements are even more challenging because the quay walls have been already used for more than 45 years which comes close to the original design service lifetime of 50 years. Furthermore, the upgrade is required to redesign the structure in such a way to lengthen the service life time with additional 50 years.

In this paper, the rebuilding will be analyzed and reasoned from risk assessments. Focus will be put on:

- How to fully characterize the initial situation and conditions.
- What are the possible risks by rebuilding?
- What are the necessary mitigation measures?
- Do some uncertainties still remain?

As the most important risks are evidently firstly technical, the financial and juridical aspects will not be directly dealt with in this paper.

2. Case Study: a Brief Description of the Structure

The case investigated here is heavy quay walls which are more than 45 years old and was designed according to a deterministic approach.

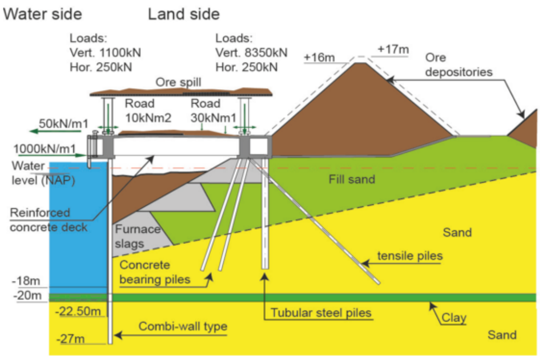

Figure 2 Schematic diagram of the case study

At this location, see Figure 2, soil investigation which was previously carried out showed that the subsoil consists of mainly sand and a 2 m thick layer of clay starting at -18 m NAP. Sand has been added to achieve the current ground surface level. The average water level is nearly NAP.

The structure consists of a combi type wall made of double PSP 600 steel piles with sheet piles in-between, see Figure 2. The foot of the sheet piles stands higher than that of the PSP -piles. The deck which is made of reinforced concrete and is part of a hollow construction where drainage systems are installed. The back of the deck is supported by 2 rows of inclined concrete piles of 0.450 m in diameter and a c.t.c. distance of 2.35 m and a row of tubular steel pipes of 1.02 m of diameter and a c.t.c distance of 5.0 m. Additionally, a row of inclined tension steel piles Pst 300 c.t.c distance 3.60 m contributes also to the stability of the whole structure.

Underneath the deck structure, dams made of blast furnace slags have been built to provide additional geotechnical stability.

The transfer of the iron core from the ships to the depositories takes place through transport trucks and conveyors. Spills of iron cores happen at difference places of the transfer. Inspection of the current structure showed that a large amount of iron spills has been piling up under the deck, between the quay wall and dams of furnace slags. The spill underneath the deck structure seem to vary in thickness along the quay wall.

3. Risks of Upgrading

The process of redesigning an existing structure is summed up in Figure 3. The to be upgraded quay wall is required to be used for at least 50 years more.

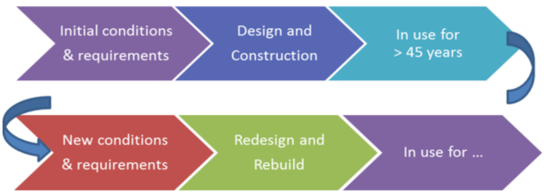

Figure 3 Process of Design-Use-Redesign

The initial (past) and new (future) designs differ in design approaches, safety philosophy and design methods as shown in Figure 4 Because of these differences, it is important to find ways to link the past and future situations.

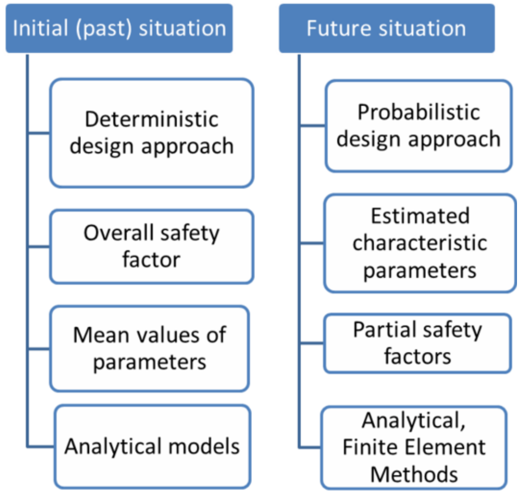

Figure 4 Characteristics of the initial and future situations

3.1. Availability of Information

The link between the initial (past) and the new (future) situations is the current one. However this link is not easy to make.

The starting points of the original design like design loads, soil and material parameters, and the used design method are known. The ultimate values of the bearing capacities of compression and tension piles are available.

Since the construction is finished, no or nearly no monitoring activities have been undertaken. No measurements were carried out at various times. Therefore none of such data are available, which could help evaluate or characterize the behavior of the structure with time. Information on the actual deformation of the structural elements related to the initial situation, especially that of the retaining wall and its bending moment is not available.

Soil field and laboratory investigations have been carried out in the past in order to determine the mean values of the parameters to be used for the design. Part of these parameters may slightly change because of the construction itself but also possibly of the impact of many years of utilization. The stress history of 45 years of use like the actually applied loads, cyclic loads, and stress paths is generally not available.

During use, it appears that there are spills of iron core at various places. The amount of these spills is not negligible as it acts as a load on the structure. Therefore they should be taken into account in the redesign of the quay wall. This information can be directly obtained by carrying out measurements. These measurements are hence available.

3.2. Unknown Aspects

Generally, the effect of 45 years of use on the material properties is difficult to determine. The extent of fatigue or/and corrosion of the material are not easy to evaluate on site.

The actual behavior of the quay wall structure is not known in comparison with the design situation. How safe and hence reliable is the current structure? What is the actual safety factor characterizing the current structure? It is clearly unknown as well as its reliability.

3.3. Methodology for Redesign

How to cope with the uncertainties of the current situation in order to be able to redesign the quay wall according to the current standards?

The method as described in Handbook Quay Walls, first edition (2005) was adopted. In this method fundamental load combinations are prescribed. The safety analysis is based on the application of overall safety factors on the bending moments and forces. The safety factors are based on a design of a new construction, with a lifetime of 50 years. By doing so, it would make possible a comparison of the structural behavior between the initial, the present and the future situations. The soil stress history should be taken into account in the new calculations of the three situations.

4. Design and Redesign

In the present paragraph, the aspects related to the design and redesign of the quay wall are considered.

4.1. Original Design

The original design was made on the basis of deterministic calculations of the different parts of the quay wall. The soil parameters were determined as the expected values based on engineering practice. Overall safety factors were used according to the standards of the original design (1970). In 2005 and 2009 additional soil investigations had taken place, consisting of CPT tests. The effect of the high loads from the ore on the soil could be established. Besides, the condition of the sheetpile wall was investigated in 2005. It turned out that some corrosion had taken place, which influences strength and stiffness of the sheet pile wall.

4.2. Redesign

The redesign (2009) was made with the FEM program Plaxis 2D (version 9.1). The use of the continuum modelling of the soil and the structure has the advantage that all different collapse criteria can be investigated in one model and that interaction between soil and structure can be modelled in a correct way. The soil parameters in these calculations were based on representative values, determined from empirical relations from the CPT tests. The calculations were done to investigate the recent and future loading situations, and to compare different measures. So these calculations could be regarded as a preliminary design stage. The definitive design of the new situations should be based on more knowledge of the soil behavior, which presumes the execution of geotechnical laboratory investigations. The design was made using partial factors for the bending moments, the anchor forces and the normal forces. This procedure is described in CUR 211, first edition. This means that the procedure was not as mentioned in Eurocode 7 (NEN-EN 9997). In 2014 the second edition of CUR 211 was published in which the design procedure was described according to Eurocode 7.

The procedure used was regarded to be valid for this construction. Although the construction already existed and was used over a period of 45 years, the construction was regarded as a new design with a lifetime of another 50 years. The value of the partial factors was determined by this assumption. In these calculations the actual stiffness of the sheet pile wall was used, corrected for corrosion, and also the effects of expected future corrosion were taken into account, the rate of the corrosion in time was estimated by comparison to other projects.
An overview of the calculations performed and the calculations steps are given in Table 1.

Table 1. Overview of calculation steps followed

STEP 1	**Original design**
1	Iron core: 22 kN/m^3 Level of bottom of waterways NAP –18 m
STEP 2	**Current situation**
2	Iron core: 31 kN/m^3 Level of bottom of waterways NAP –18 m Gained ore spills/ material deposition on slope of dam, underneath deck
2a.	*Determination of effects of accumulated ore spills / silt deposition underneath deck* Current situation (iron core: 31 kN/m^3) where ore spills and material deposition are removed
2b.	*Determination of effects of increase of vol. weight of iron core* Original design where iron core is 31 kN/m^3 (no ore spills nor deposition underneath deck are taken into account)
STEP 3	**No adjustment of the current situation for deepening waterways from NAP - 18 to –20,0 m**
3	The current level of the bottom of the waterways is deepened until NAP –20,0 m. Here a difference is made between: 1. During the construction (deepening) and 2. End situation (including material deposition until NAP –0,7 m)
STEP 4	**Transformation of the quay wall using additional anchors***
	Before the deepening of the waterways, the quay wall is reinforced using anchors at the rear of the deck. Follows the deepening of the waterways. Here, a difference is made between 1. During the construction (deepening) and 2. The use periode (including material deposition (silt 12 kN/m^3) until a depth NAP –0,7 m) Various variants have been investigated whereby the slope of the anchors and the pre tension stress were varied.
4a.	Ca. 0 deg. (shallow anchor) with pre tension stresses of 400, 500 of 600 kN/m’ were applied
4b.	15 deg. with pre tension stresses of 400, 500 of 600 kN/m’ were applied
4c.	30 deg. with pre tension stresses of 400, 500 of 600 kN/m’ were applied

Control values are needed to check the calculated values in order to design the new quay walls. These values can only be estimated from

FEM calculations and available information of the current situation. FEM was applied to take into account stress history in the structure starting from the initial till the current situation.

The various control values are taken as the calculated stresses in the structural elements in the current situation. A sensitivity analysis has been performed based mainly on the variation of the iron core volumetric weight. The obtained control values have been checked also with similar studies carried out in the same area. The obtained control values are presented in Tabel 2. In table 2, other control values are given like the yield stresses and the maximum bearing capacities.

Table 2. Estimated control values from FEM calculations

Structural element	**Yield stress** [N/mm^2]	**Control value stress in material** [N/mm^2]	**Max. Bearing capacity (design value)**
Peiner wall PSP 600 (ctc 1.66 m)	360	120 (=360/1.3)	-1742 kN/m'
Tension piles (ctc 3.62 m)	240	184 (=240/1.3)	564 kN/m' *[Iron core: 31 kN/m^3]*
Tension piles (ctc 3.62 m)	240	184 (=240/1.3)	524 kN/m' *[Iron core:22 kN/m^3]*
Tension piles (ctc 3.62 m)	240	184 (=240/1.3)	423 kN/m' *[No iron core in deposit.]*
Conc. bearing piles (ctc 2.45 m)	n.a.	-	- 2170 kN/m'
Conc. bearing piles (ctc 5.0 m)	n.a.	-	- 1416 kN/m'

From the outcome of the performed calculations, the stability of the construction turned out to be partly depending on the stiffness of the blast furnace slags underneath the deck structure. As this material turned out to be cemented during the life time of the quay wall, it was not possible to take out samples and investigate the stiffness and volumetric weight.

In the calculations the complete loading history, including the construction of the quay wall was modelled.

From the calculations with the enlarged depth and increased ore loads it turned out that the capacity of the sheet piles was sufficient, but the capacity of the tension piles was not sufficient. Besides, it turned out that because of consolidation settlements must have occurred in the past. This introduces bending moments in the tension piles, which are more or less embedded in the concrete deck construction. In the original design this effect was not taken into account. This shows the advantage of a FEM model, this possible failure mechanism was apparently overlooked during the design.

Several load combinations were investigated in the Plaxis model, using different partial factors on the loads. It turned out that the combination in which the all partial factors were equal to 1.0 led to the maximum bending moments and anchor forces.

In the Plaxis model measures were investigated to encounter the overload of the existing anchor piles. The fact that the quay wall has to remain in function and disturbance of the production process is hardly possible, made it difficult to find a good solution. The most attractive solution was found in the application of rather shallow extra anchors, placed from inside the deck construction. In Figure 5 the calculated deformed mesh, in the stages after deepening and application of high bulk loads is shown. In this figure also the bending of the original tension piles as well as in the concrete piles is clearly shown.

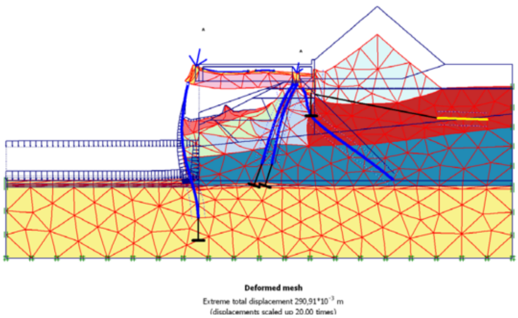

Figure 5 Deformed FEM mesh, final stage

The deepening of the harbor and the increase of the load also result in higher forces in the concrete deck construction. To ensure the life time of 50 years the structural capacity of the reinforcement turned out to be insufficient. Measures that were proposed are the application of extra reinforcement underneath the deck.

From the calculations it was concluded that both the bending and bearing capacity of the sheetpile wall remain sufficient after the redesign.

Also the safety regarding the passive earth pressure was sufficient.

4.3. Remaining Risks

The calculation method as described above does not completely fulfill the requirements of Eurocode 7, but was considered as the best available at the time of the preliminary redesign for the case of a heavy loaded quay wall. Since 2014 the regulations as formulated in the second edition of the Quay Wall handbook fully comply with the Eurocode system. So the final design can be completed with this method.

One of the remaining uncertainties in the design concerns the uncertainty in the soil parameters. In this case the properties of the blast furnace slag are probably dominant. A thorough investigation of this material is necessary to determine the stiffness and strength.

Besides the effects of corrosion on the properties of the sheet pile wall are unknown. Measurements were executed in 2005. With time the corrosion will increase, unless measures like cathodic protection are used.

Because of these remaining uncertainties, efforts were put, at that stage, to optimize the redesign by carry out a limited sensitivity analyses using FEM calculations of the additional anchors. In this analysis the influence of the stiffness and strength of the soil parameters and of the inclination of the anchors were regarded. It was concluded that anchors have to be designed at a tension force of 600 kN/m' and places under an angle of 30 ˚ with the horizontal surface. In this case the force in the tension piles will be reduced to an acceptable level, so the combined tension and bending stresses will fulfill the strength requirements.

5. Conclusions and Recommendations

From the study that was performed for the case of an existing heavy loaded quay wall the following conclusions were drawn.

1. The design codes based on Eurocode 7 are valid for newly built constructions, but do not incorporate the analysis of existing constructions. Although a new code dedicated to the assessment of existing geotechnical structures is in preparation, it was not available at the time of the carrying out of the feasibility study.
2. The FEM method revealed a failure mechanism that was not taken into account in the deterministic approach of the original design. For complex geotechnical constructions, the FEM method is highly advised.
3. The procedure of redesign is hampered by the lack of reliable knowledge of the effects of the use and of the behavior in the past.
4. Monitoring of newly built important infrastructures is vital to be able to determine the remaining strength of the structures after their design lifetime is exceeded.

Based on the above-mentioned conclusions, it would be recommended for the final design of the upgrade:

- To carry out an extensive sensitivity analysis, especially on the material parameters.
- To carry out a reliability analysis .
- To perform 3D calculations to optimalise the behavior of soil around piles.
- To monitor the structure deformation during and after the upgrade and to check these with the design values, more specifically to apply the observational method.

In general, a redesign of an existing and complex quay wall would necessitate an extensive design procedure to increase reliability and therefore to create acceptance of risks.

Acknowledgement

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