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Check of caisson stability using 3D EE modelling
Vérification de la stabilité d'un caisson par une modélisation 3D EE

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ABSTRACT: For the construction of a ten year old quay wall at Antwerp concrete caisson wth a diameter of about 30 meter have been used. For the control of the overal stability of the caissons calculations have been performed with the FLAC 3D program. Due to the special type of construction (concrete caisson filled with sand), several problems occured.
From the performed calculations it could be deduced that :
- The calculated deformations are similar to the measured deformations.
- The overal stability is influenced by many parameters and is not easy to be determined.

RÉSUMÉ: Pour la construction d'un mur de quai il y a 10 ans à Anvers, des caissons de béton d'un diamètre d'environ 30 mètres ont été utilisés. Pour le contrôle de la stabilité générale des caissons, des calculs ont été réalisés avec le programme FLAC 3D. Du fait du type spécial de construction (caisson de béton rempli de sable), plusieurs problèmes sont survenus.
Les calculs réalisés permettent de déduire que:
- Les déformations calculées sont semblables aux déformations mesurées.
- La stabilité générale est influencée par de nombreux paramètres et n'est pas déterminée facilement.

1 INTRODUCTION

In 1987 started the construction of a quay wall as Container Park in the Seaport of Antwerp.

The quay wall is situated along the river Scheldt with a tidel water level variation of 4 to 5 meters. The quay wall was carried out using the sinking technique of several concrete made cylinders (caissons) with a 29 m outer diameter, 30 m height and 0.95 m thickness.

Sinking was performed in dry conditions. To make the sinking easier, a 10cm ring of bentonite based grout was placed along the outer surface of the caissons.

An opening of about 1 m was left between two adjacent cylinders and jet-grout columns have been installed in this opening in order to prevent erosion through these openings. Afterwards a concrete retaining wall and a cone shaped enbankment have been installed in front of the openings between the caissons.

For the design of the quay wall only 2 dimensional simulations have been carried out. As some remediation works had to be performed to limit erosion through the joints between the caissons, it was decided to check also the overall stability of the quay wall using a 3 dimensional simulation with new programs which recently appeared to perform more accurate 3 D-modelisations of complex structures.

The study of the overall stability of the quay wall was done in association with the studies office

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Jan Maertens BVBA, SBE and Tractebel Development Engineering. The present study is performed with the FLAC^{3D} Finite Differences program.

The objective of the modelisation is the assessment of the soil-structure displacements during the sinking of the caissons and for the final sitation. The global factor of safety will be estimated. The displacement at the head of the caissons and the stresses under the caissons cutting edge will be investigated more particularly.

2 FD MESH AND MODEL STRUCTURE

2.1 *Soil characteristics*

The following sketch shows the soil configuration in which the caissons have been sinked. In the model, the soil characteristics have been reduced till 5m behind the quay wall to allow for decompression during the sinking process.

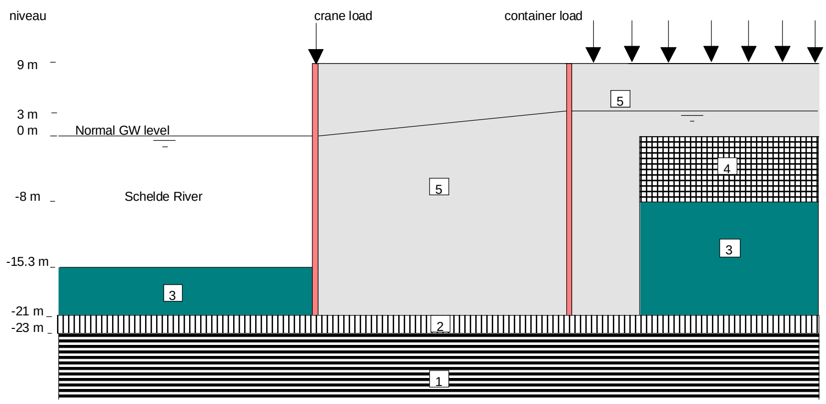

Fig 1 : Soil layers configuration

Layer n°	Soil description	Level	γd kN/m³	γn kN/m³	φ' °	c' kN/m²	ν	G kN/m²	E kN/m²
		-35.00							
1	Compact sand		17	20	35	1	0.3	87.5	227.5
		-23.00							
2	Sand of Kruisschans		17	20	27	10	0.3	45	117
		-21.00							
3	Sand of Merksem		17	20	35	1	0.3	65	169
		-8.00							
4	Kwartaire afzetting		17	20	30	1	0.3	50	130
		0.00							
5	Back filling		17	20	27	1	0.3	25	65
		9.00							
6	Concrete		25	25			0.2	13600	35360
7	Grouting column		25	25			0.2		10000

Fig 2 : Soil layers properties

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2.2 *Sequential modelling following the different construction phases*

The modelisation of each construction stage allows to relate the actual soil stresses history around the caissons:

1. Initial situation with the water table lowered to -21.0m
2. Sinking of the caisson in three stages until -21.0m and ground characteristics reduction around the structure;
3. Backfilling of the caisson in three stages
4. Carrying out of the upper structure until +9.0m and backfilling inside and behind this one;
5. Stopping of the ground water lowering;
6. Carrying out of the jet grouting column between two adjacent cylinders and dredging in front of the quay wall in two stages ;
7. Loads application : - load due to the containers wrecking crane (70 T/m)
   - load due to stocking of the containers (20 kN/m²)

2.3 *Tractebel model*

The figures on the next page show the actual geometry of the caisson wall and the final grid mesh geometry of the 3D model. Particular attention has been paid to simulate as good as possible the actual geometry of the quay wall and specially the different stability elements carried out to constitute the joint between two adjacent caissons (jet grouting column, gravel cone, retaining wall of the embankment).

Owing to the large dimensions of the quay wall and the necessity to keep a grid mesh as regular as possible for numerical reasons as well as for accuracy reasons in the region of interest, the number of elements and nodes reach rapidly great value. The designed model is made of more than 22000 elements and 25000 nodes. The calculations are carried out in total stresses and the actual water table levels are introduced as well as their variations during the different stages of the construction.

The pore water pressures are then fixed and the effective stresses can be deduced at each calculation stage.

The possibility to simulate the soil-structure contact along the walls of the caisson through “interface” elements has not been kept owing to the large calculation time induced by the great number of contact faces on the internal and external parts of the caisson.

In these conditions the calculation time per stage of modelisation reached in average 15h on a 200 MHz Pentium PC.

2.3.1 *Results*

For the existing situation, the calculated total horizontal displacements at the top of the caisson reaches about 2cm.

The discrepancy with the observed movement of about 5cm may be attributed to the friction alongside the external wall of the caisson which is overestimated since no interface elements are introduced to simulate the presence of bentonite set up during the sinking.

At this final stage, an evaluation of the safety factor is carried out reducing the ground characteristics ϕ’ and c’. According to the partial safety factors recommended in Eurocode 7, the tangent of the friction angle $tg\phi$’ is reduced by a factor 1.25 and the cohesion *c*’ by a factor 1.6.

The results show that the resulting security is available since the model reaches the equilibrium state. The horizontal displacement at the top of the caisson increase to about 7.5cm. Nevertheless further reduction of ground characteristics yields to instability.

Dewatering behind the caisson from the normal water level (+3.5m) to -2m at the time of a first stage and then to -5m constitutes a additionnal factor of stability and will give rise to a higher safety factor.

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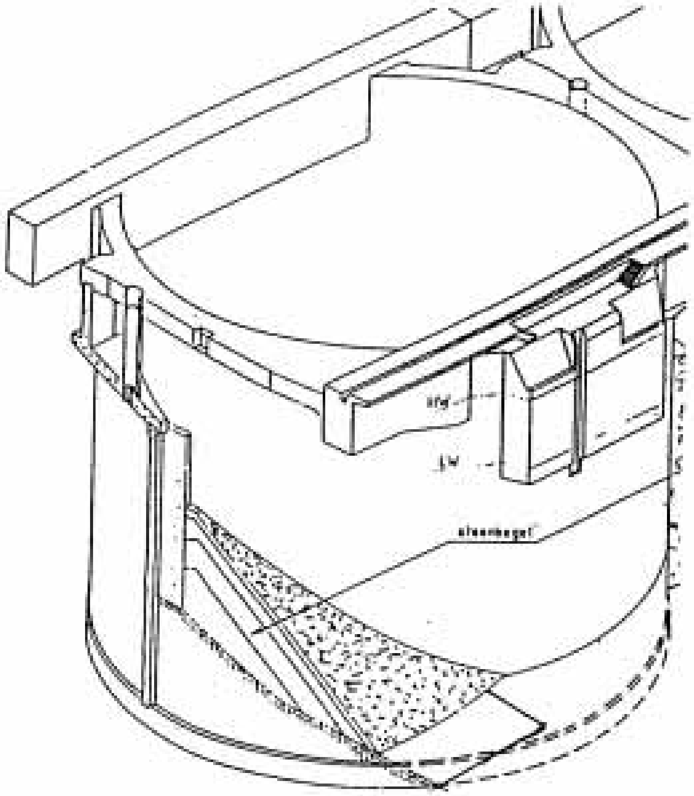

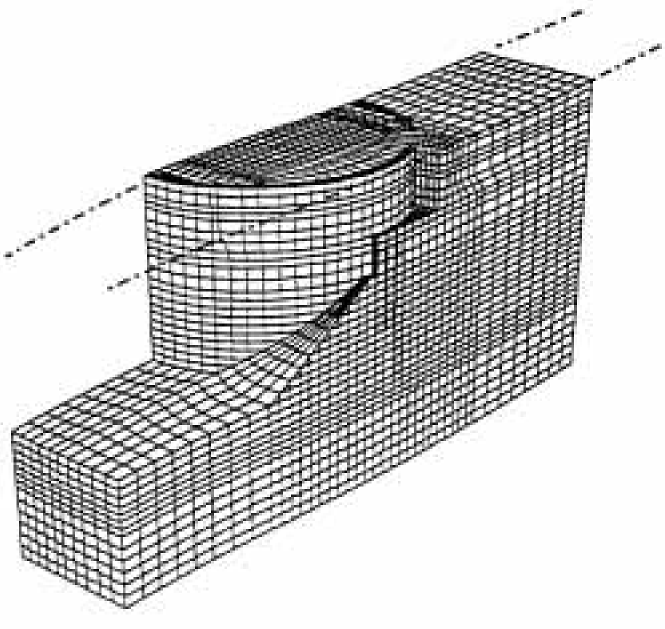

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2.4 *SBE model*

In a first phase it was the intention to model a set of tree caissons, a middle caisson complete and two neighbouring caissons half, to simulate different soil characteristics and differential movement. Because of model size limitations it was obvious that such complex model was not realistic and the model was limited to one complete caisson.

In building the model the concrete caisson was modelled without detail in the superstructure because the concrete structure was of minor importance over the soil-structure interaction. Also the rockfill “domes” between the caissons placed a few years ago in an attempt to repair the joint leaking was modelled quite schematically as a pyramid of soil elements with a shape approaching the real shape.

In first versions of the model the mesh size was adjusted to reach an acceptable level of accuracy within reasonable calculation times. By gradually reducing the mesh size near important regions like the bottom of the caisson it was possible to obtain high accuracy where needed. The model result was compared to a 2D-mesh with high accuracy but shorter calculation times to compare the results and check for possible errors.

To model the existing bentonite residue on the outside of the caisson it was necessary to introduce interface elements which allow slip between soil and caisson to occur. Due to the very strong impact of these interfaces on the calculation times only an outside interface was modelled. The fill inside the caisson was considered to be accurately without an interface concrete-soil.

2.4.1 *Different construction phases and the way they were introduced in the calculations*

To limit the complexity of the calculations the construction history of the caissons was started on the moment when the caissons were already in place and on level. The actual sinking of the caissons was not included because it has no big influence on the surrounding soil pressures. The following phases were handled in the model:

1/ soil pressure calculation of general model - caissons empty - groundwaterlevel - 20.
2/ gradually filling the caissons in a few steps.
3/ filling in and behind the caisson to present soil level.
4/ rise of the water table to its present situation.
5/ dredging of the soil in front to present dredge depth
6/ bollard forces, crane and live load.

2.4.2 *General Results*

Existing situation

Concerning the existing situation it could be shown that the calculated deformation of about 6 cm showed good agreement with the measured deformation of about 5 cm. On the other hand the iterative solution progress of the calculation went very slowly and stabilised poorly, which showed that the stability of the model (and thus likely the reality) was poor.

The soil pressure diagrams showed a big difference between the front and back end of the caissons, which is the result of the large eccentricity of the forces acting on the bottom of the caisson. The high soil pressure under the front “knife” of the caisson is one of the main reasons of the poor stability of the caissons. A clear vertical displacement could be found under the knife of the caisson front, and an overall tilt of the caisson was the result.

Phi-c reduction

To calculate the "factor of safety" of the structure a phi-c reduction was performed. This is a calculation technique in which the soil characteristics of the model are gradually lowered in steps. In every step the model is recalculated until failure appears. The reduction of the soil characteristics by which this failure happens gives a measure for the factor of safety.

Because of the long calculation times and the slow convergence of the iterative calculation the planned phi-c reduction could not be performed in this way, and it was not possible to set a clear factor of safety.

Phi-c reduction in one step

To achieve a measure for the factor of safety a direct approach was introduced to the phi-c schedule. By reducing the soil characteristics in one step to a required safety factor (required 1.25) and performing a calculation it was possible to quickly obtain a result from the program. When the iterative process showed (slow) convergence it could be stated that the model was stable. If the process showed no convergence at all and a linear displacement of the caisson, it was obvious that there was no solution so the caisson was not stable anymore. In this way it was possible to estimate a maximum safety factor.

The calculation with a reduction of 1.25 on tan(phi) and 1.6 on c showed no convergence at all. This proved that the real safety of the caisson was not within normal design practice.

Dewatering behind the caissons

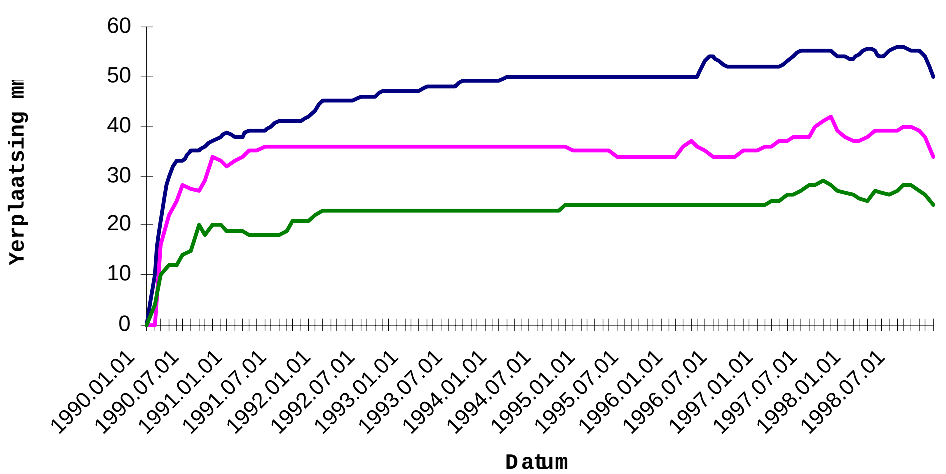
The phi-c reduction in one step made it possible to quickly analyse the result of a permanent dewatering behind the caissons. By lowering the water table the water pressure from the Scheldt river becomes a stabilising factor for the caissons.

The normal water level behind the caisson is about +3.5 TAW. Tree levels of dewatering were introduced -2, -5 and -8 TAW. The calculations showed that -2 did not stabilise the caissons yet with the normal factors of safety, -5 and deeper showed quick convergence to a stable solution.

3 MEASUREMENTS IN SITU

Following the calculations the construction of a dewatering tunnel was started and is now in a final stage. The dewatering which was neccessary for the construction of the tunnel was followed-up by measurements of the caisson movements by means of inclinometers. It is clearly visible that due to the groundwater lowering the caissons move backwards by several mm. The following graph shows the horizontal displacement on groundlevel, level -10 and level -20.

Inclinometer caisson 40



4 CONCLUSIONS

By performing the FE calculations the following results were achieved :

- It was possible to perform the general modellisation of the existing structure in the present situation concerning water levels, soil characteristics and the relation of the soil characteristics near the concrete caisson with the building history. The deformations obtained in the model calculations are comparable to the deformations measured in situ.

- In general it could be shown that the existing structure is stable, although with a limited and clearly too small factor of safety.

- An exact calculation of the factor of safety was not possible due to the complexity of the problem and the extra-ordinary calculation times. However, it was possible to deduce that the maximum factor of safety was less then required for the design of this retaining wall compared to the normal design practice in Belgium, and that bigger and non-stabilising deformations could be expected in the future.

- The model showed clearly that the effect of a permanent dewatering behind the caissons has an important effect on the overal stability and proved an efficient means of improvement.

5 ACKNOWLEDGEMENTS

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REFERENCE

H. Smitz, Proc. 9th Intl. Harbour Congress Antwerp (1988) : *Riverside Container Quay on the Scheldt.*