Proceedings of the XVII ECSMGE-2019
Geotechnical Engineering foundation of the future
ISBN 978-9935-9436-1-3

doi: 10.32075/17ECSMGE-2019-0247

XVII European Conference
ECSMGE 2019
Reykjavik
on Soil Mechanics
and Geotechnical Engineering

MV tension pile load tests in the Port of Rotterdam: practical aspects and geotechnical behaviour

Essais de charge de pieux tendus MV dans le Port de Rotterdam: aspects pratiques et comportement géotechnique

J. Putteman
MariTeam/SBE Engineering Consultants, Sint-Niklaas, Belgium

E.J. Broos, H.E. Brassinga
Port of Rotterdam Authority, Rotterdam, the Netherlands

R. Spruit
Municipality of Rotterdam, Rotterdam, the Netherlands

M. De Vos
Belgian Building Research Institute, Brussels, Belgium

A.L.J. Timmermans
MariTeam/B.V. Ingenieursbureau M.U.C., Terheijden, the Netherlands

ABSTRACT: This paper describes the practical aspects and results of 2 recent tension pile load testing programmes in the Port of Rotterdam. The pile load tests are executed in the framework of the construction of 2 major deep sea quay wall projects. Because of the exceptional pile lengths up to 60m these grout-injected MV piles (HEB600 steel profiles) were subjected to an extensive testing programme to validate the design skin friction. In total, 11 static load tests were executed. For each pile the strain is recorded by fibre optic sensor lines along the entire pile length. The piles were designed with the hypothesis that their geotechnical resistance is solely provided by the soil outside the active soil wedge. Measures were taken to debond the pile from the surrounding soil in the active soil wedge, which proved to be very difficult in practice. The practical aspects related to debonding are discussed in detail. The measured skin friction is compared to historic test results.

RÉSUMÉ: Ce document décrit les aspects pratiques et les résultats de deux programmes d'essais récents de charge de pieux tendus dans le Port de Rotterdam. Les essais de charge de pieux sont exécutés dans le cadre de la construction de 2 grands projets de murs de quai en eau profonde. En raison des longueurs de pieu exceptionnelles de 60 m, ces pieux MV injectés de coulis (en acier HEB600) ont été soumis à un programme d'essais approfondi pour valider le frottement. Au total, 11 tests de charge statique ont été exécutés. Pour chaque pieu, la déformation est enregistrée par des capteurs à fibre optique sur toute la longueur du pieu. Les pieux ont été conçus avec l'hypothèse que leur résistance géotechnique est uniquement fournie par le sol situé à l'extérieur du plan de rupture. Des mesures ont été prises pour délier le pieu du sol environnant dans la partie de sol en état actif, ce qui s'est avéré très difficile en pratique. Les aspects pratiques liés au décollement sont discutés en détail. Le frottement mesuré est comparé aux résultats des tests historiques.

Keywords: MV pile; tension pile; pile load test; deep sea quay wall

1 INTRODUCTION

Since the first application in the Port of Rotterdam in 1982, MV (Müller Verfahren) tension piles are frequently used as horizontal anchorage of deep sea quay walls in Rotterdam, as they combine a high bearing capacity with a good resistance against corrosion and ship collision. The MV piles consist of a steel beam and a grouted cover. During driving of the steel beams grout is supplied by 2 grout pipes leading towards reservoirs mounted at the toe of the pile (Figure 1). The shell of grout that is formed around the pile is advantageous for the driveability and - after hardening - also for shaft resistance.

Figure 1: MV pile tip with reservoir and grout pipe

The original MV pile design developed in Germany was optimised for the application in the Rotterdam area during the 1980s. A reduced cross section of the grout reservoirs results in reduced pile driving resistance and less grout consumption.

In early years, the required characteristic pile bearing capacity was limited to 4000 kN, and steel beams e.g. PSt 370 (de Gijt and Brassinga, 1990) or PSp 602 (Brassinga, 1987) with pile length up to 35m were used. A major step towards increased bearing capacity was taken in 2005, when HEB 600 steel beams with pile length up to 56m and a characteristic geotechnical bearing capacity of approx. 11000 kN were installed for the first time (van Paassen & van Dalen, 2009).

Recently an extensive pile load testing campaign was undertaken in the framework of the construction of 2 new deep sea quay walls at the Port of Rotterdam, i.e. the Offshore Terminal Rotterdam (referred to as OTR, construction period 2016-2017) and the Hartel Tank Terminal (referred to as HTT, construction period 2018-2019).

2 QUAY WALL DESIGN

The design of both quay walls is fairly similar as they are both designed for a retaining height of approx. 30m and the subsoil conditions are comparable. Each quay wall consists of multiple equal size segments, each 23m long. Each segment basically consists of a superstructure and a substructure (Figure 2).

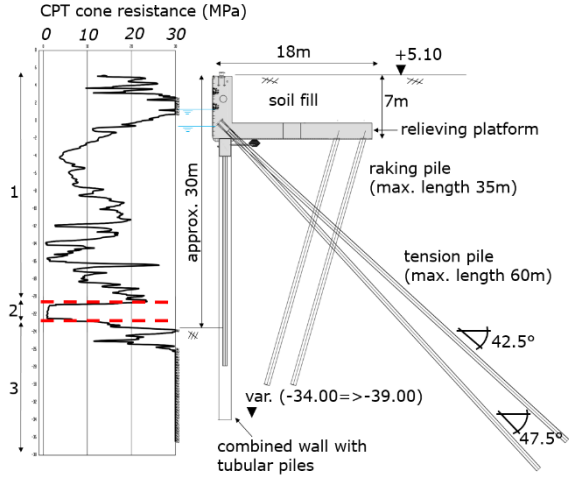

Figure 2: Offshore Terminal Rotterdam. Left: CPT (Cone Penetration Test) soil profile. Right: Cross-section of the deep sea quay wall. The tip levels of the tubular piles vary due to geological variations. The CPT and the cross-section are drawn on the same scale and refer to the same level.

The superstructure is a L-shaped concrete relieving platform. As the relieving platform is carrying more than 5m of soil fill, the lateral earth pressure on the combined wall is largely reduced. The use of a relieving platform is absolutely necessary in order to minimize the combined wall dimensions and guarantee the constructability, particularly with regard to the heavy pile-driving work and related soiltightness of the wall.

The substructure has a soil retaining function and serves as the foundation of the superstructure. The substructure consists of:

- Combined wall. The primary elements are high quality X70 steel tubular piles with diameter 1420mm and plate thickness up to 24mm. Each primary element is connected to a secondary element (triple PU 28 sheet pile) by means of interlocks.
- Compressive raking piles (batter piles) inclined at an angle between 9 and 16 degrees. Because of the high resistance of the Pleistocene sand layer and expected heavy pile-driving, after extensive investigations (driving analyses) it was concluded that a screwed tubular pile type with permanent casing (diameter 609mm), grout injection and a lost screw tip (diameter 850mm) is most appropriate.
- MV piles with a maximum length of almost 60m inclined at alternating angles of 42.5° and 47.5°. The exceptional length required a purpose-built piling rig (Figure 3) with a leader length of 70m. The piles are driven with a IHC S120 hydraulic hammer.

Based on the soil profile presented in Figure 2, distinction is made between the following soil layers:

- 1: Antropogene and Holocene sands, moderate to dense. The major part of this layer is hydraulic fill from the construction of the Maasvlakte reclaimed area.
- 2: Wijchen clay layer.
- 3: Pleistocene sands, very dense, characterised by Cone Penetration Test (CPT) resistance values exceeding 50 MPa.

The design of the MV piles is based on historic test results. In the 1980s several pile load tests have been executed from which the following design rule for the characteristic shaft resistance in the Rotterdam harbour area was derived (de Gijt & Brassinga, 1990):

$$q_s = 0.014.min(q_c, 18MPa) \quad (1)$$

$$q_{s,max} = 0.014.18MPa = 252kPa \quad (2)$$

$$R_s = c.\int_0^L q_s.dz \quad (3)$$

Where q_s (kPa) is the characteristic skin friction, q_c (MPa) is the CPT cone resistance, c (m) is the circumference defined by the rectangular contour line around the reservoirs near the toe (represented by the dashed red line in Figure 4 (Brassinga, 1987)), L (m) is pile embedment length and R_s (kN) is the characteristic shaft resistance.

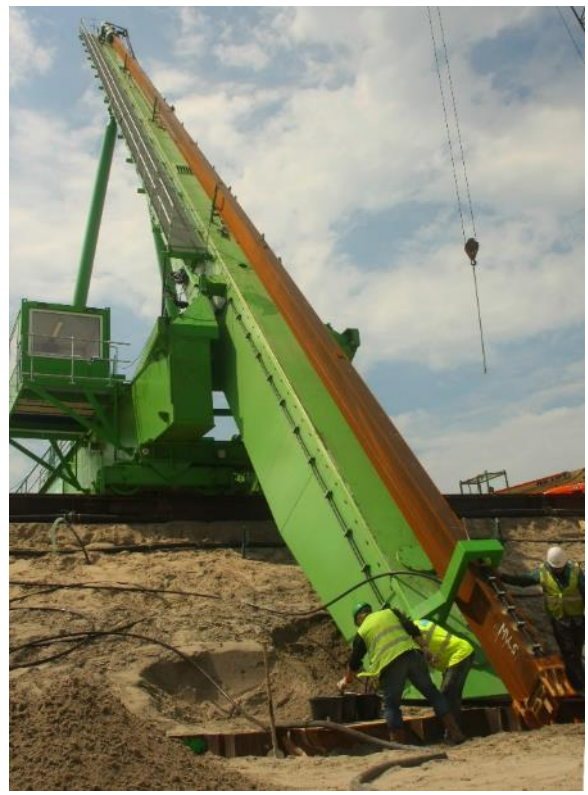

Figure 3: Piling rig

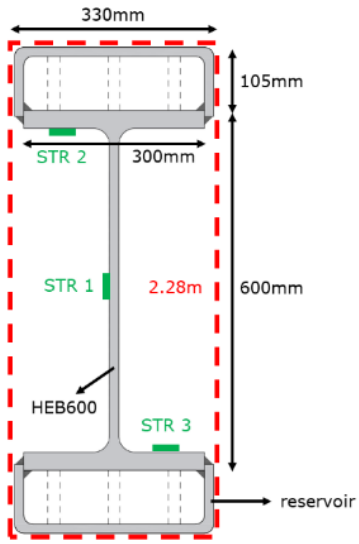

Figure 4: MV pile with grout reservoirs and a circumference of 2.28m (dashed red line). The position of the strain gauges is indicated by the green rectangles.

3 Test setup

The ultimate goal of the test campaign was to verify the maximum skin friction of 252 kPa which was used for the design of the quay wall by several instrumented static load tests.

The piles were designed with the hypothesis that their geotechnical resistance is solely provided by the very dense Pleistocene sand layer underneath the Wijchen clay because of the presence of the active soil wedge above the clay layer, as shown in Figure 5. Therefore the testing campaign aimed at mobilizing the maximum skin friction in the very dense sand layer, which is expected to be achieved at 2 mm creep rate (failure, according to (CUR 166, 1997)). The creep rate is defined as the slope of the pile head displacement curve plotted on a logarithmic timeline. For this reason all test piles are debonded from the surrounding soil by means of an oversized casing, which is driven immediately after driving the MV test pile. The soil and grout inside the casing are flushed out by hydrojetting.

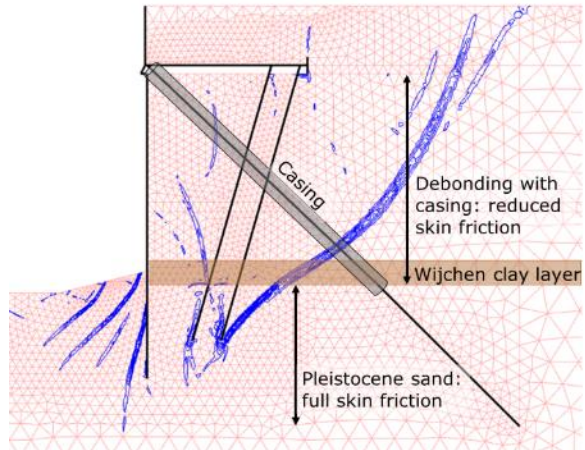

Figure 5: Visualization of shear planes in the finite-element model of the OTR quay wall. For the loading test the MV pile is debonded in the active soil wedge above the Wijchen clay layer by means of an oversized casing.

The pile test load is limited to 95% of the yield strength of the steel beam. For a yield strength between 355 MPa and 420 MPa the maximum test load thus varied between 9100 kN and 10770 kN. The maximum test load is reached after approx. 10 load steps of each 30 resp. 60 minutes as shown in Figure 6. To allow for the mobilisation of the maximum skin friction in the Pleistocene sand layer, a reduction of the skin friction above the clay layer by using an oversized casing is of utmost importance to have a maximum tensile load available below the clay layer in the Pleistocene sand.

Both for OTR and HTT a purpose-built steel reaction frame was used (Figure 7) for transferring the tensile load from the test pile in the centre of the frame towards the 2 outer reaction piles. The centre-to-centre distance between the test pile and the reaction pile amounts to 5.76m (OTR) resp. 6.59m (HTT). The tensile load is generated by 4 hydraulic jacks.

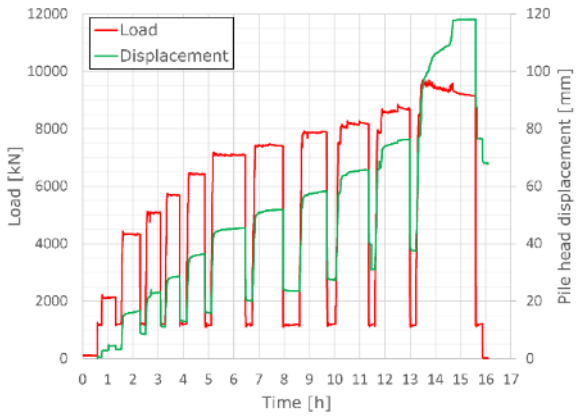

Figure 6: Load and pile head displacement of pile HTT1 versus time.

Each test pile is equipped with 3 fibre optic sensor lines along the entire pile length (Figure 4) to measure the axial strain and to gain better insights into the geotechnical behaviour of the pile. The majority of the fibre optic sensors are glued to the steel beam before pile driving. However, a few sensors were installed after pile driving by lowering the fibre into a steel tube which was welded to the pile and by fixing the fibre inside the tube with a grout mixture. The skin friction along each pile section can be calculated from the strains measured by the fibre optic sensors, so it is possible to distinguish between the skin friction in the active soil wedge and the skin friction in the Pleistocene sand layer. In the Pleistocene sand layer the strain is measured at intervals of 1 or 2m along the pile (OTR) resp. 25cm (HTT). The measured strains (and resulting axial force and skin friction) are relative to the state of the pile after pile driving.

Figure 7: Reaction frame at Hartel Tank Terminal

4 OTR TEST RESULTS

At OTR 5 pile load tests were performed, all test piles are debonded above the clay layer with an oversized casing. Initially it was planned to use permanent piles which could be finally incorporated into the concrete superstructure and serve as the anchorage of the quay wall. However, during the test campaign it proved to be very difficult to achieve sufficient debonding of the pile and to mobilise skin friction. For this reason, for test no. 4 and 5 it was decided to use temporary piles which are significantly shorter. It was expected that these piles could eventually be loaded to failure because of their lower bearing capacity. Figure 8, Figure 9, Figure 10 and Table 1 illustrate the measures that were taken to debond the pile from the surrounding soil above the Wijchen clay layer. From Table 1 and Figure 11 it can be concluded that some of the debonding measures were more effective than others. In practice it proved to be difficult to flush the entire casing length. Piles OTR1 and OTR3 transfer approx. 5000 kN to the casing. The measures taken for piles OTR2, OTR4 and OTR5 were more successful, transferring max. 2600 kN to the casing. The use of a casing which is driven with simultaneous bentonite injection along the inside and outside perimeter is most successful.

In these tests, it was found that only pile OTR5 was loaded close to geotechnical failure – this was the only test in which the creep criterion was

decisive, and not the structural strength of the pile or reaction frame. All previous tests had to be suspended to prevent loading the steel piles and reaction frame to structural failure.

Table 1. Measures taken to debond the test piles from the surrounding soil, the resulting tensile load transferred to the casing and the mean skin friction in the Pleistocene sand layer

#	Measures taken	kN to casing	q_{savg} kPa
1	Casing Ø 813mm + flushing	4519	99
2	Casing Ø 813mm + flushing, soil remoulding using a drilling rig for mixing the soil with bentonite along the casing circumference	2603	161
3	Black varnish coating on pile and inside casing Ø 813mm + flushing.	5398	102
4	Black varnish coating on pile and inside casing Ø 813mm + flushing, casing driven with bentonite. Shorter pile.	2174	299
5	Same as 4, but larger casing diameter (Ø 1016mm), even shorter pile and pile loaded to failure.	1788	408

Figure 8: Crack in the ground caused by excessive load transfer from the test pile to the casing

Figure 9: Black varnish coating as a debonding measure

Figure 10: Pile tip of the oversized casing with bentonite injection

The mean skin friction in the sand layer is calculated as:

$$q_{s,avg} = \frac{F}{c.L} \tag{4}$$

Where $q_{s,avg}$ (kPa) is the mean skin friction, F (kN) is the pile load at the top of the sand layer, c (m) is the pile circumference and L (m) is the embedment length in the sand layer.

The maximum skin friction is obtained for OTR5 as this pile shows the highest load decrease per meter depth in Figure 11. A mean skin friction of 408 kPa along the embedment length in the very dense sand was measured in the final

load step, which is significantly higher than the 252 kPa characteristic unit shaft friction used in the design. For this reason it was decided to conduct additional load tests in the framework of the HTT project, based on the test setup of pile OTR5.

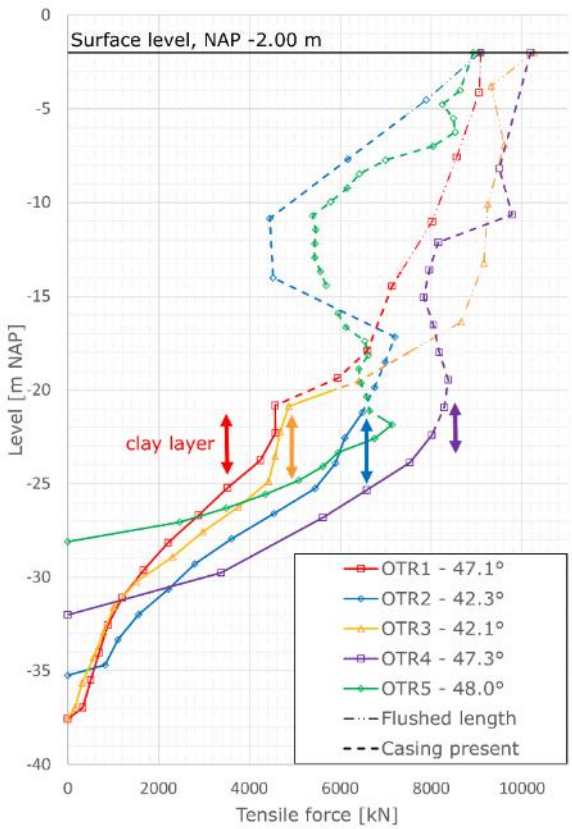

Figure 11: Results of the OTR pile testing campaign: Tensile force vs. depth at the final load step. Solid lines indicate the pile sections where no measures to debond the pile from the surrounding soil were taken. At pile OTR5 the clay layer is locally absent. The Pleistocene sand layer is located below -24 m NAP. The flushed length for OTR4 and OTR5 is unknown.

5 HTT TEST RESULTS

6 temporary piles were loaded and the same debonding measures as for OTR5 were used. The calculated pile length was based on the previously obtained mean skin friction of 408 kPa and a load transfer to the casing of almost 3000 kN in order to achieve pile failure in the last loading step. The variation of the tensile force with depth is shown in Figure 12. For pile HTT1 the load transferred to the casing amounts to 2200kN, but for HTT2, 3 and 4 it was higher than anticipated with 4500kN resp. 3400kN resp. 3900 kN. Nevertheless all piles were loaded to 2mm creep.

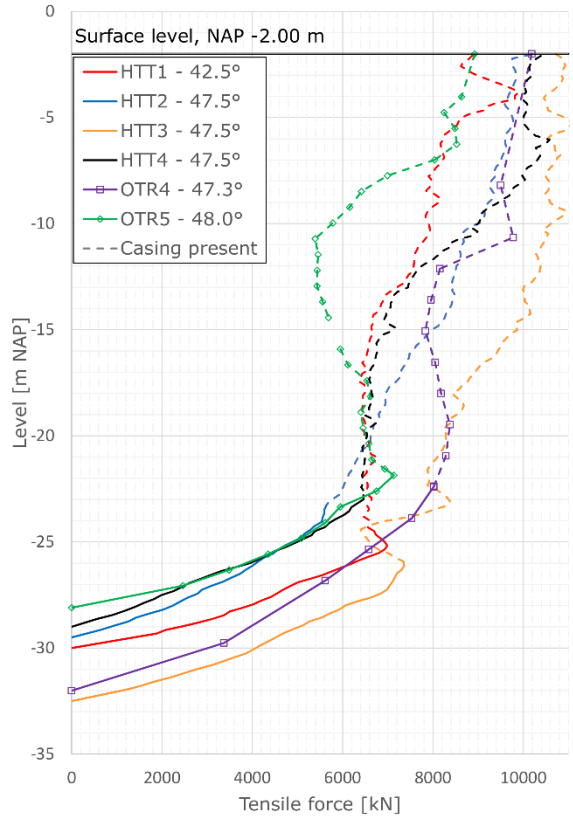

Figure 12: Results of the HTT pile testing campaign compared to OTR4 and OTR5: Tensile force vs. depth at the final load step. Solid lines indicate the pile sections where no measures to debond the pile from the surrounding soil were taken.

From Figure 13 it appears that the shape of the tensile force vs. depth curve becomes concave with increasing load, which means that the measured local skin friction increases with depth in the Pleistocene sand in the final load step, although the sand layer is quite homogeneous. As shown in Figure 14 the skin friction ranges between 250

kPa at the top of the Pleistocene sand layer towards 600 kPa near the toe of the pile. Further analysis of the load test data will eventually lead to a new CPT-based design rule for MV tension piles in the Rotterdam harbour area.

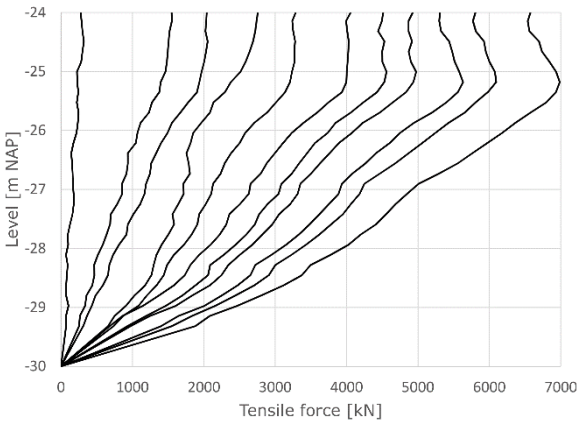

Figure 13: Tensile force vs. depth for the 11 load steps of pile HTT1

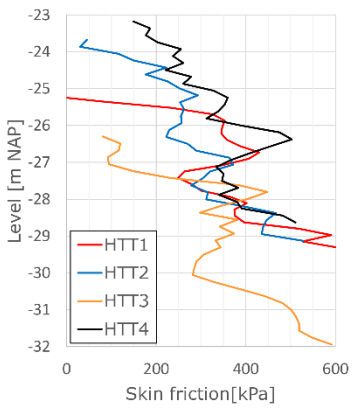

Figure 14: Measured local skin friction vs. depth in the Pleistocene sand for the HTT piles

6 CONCLUSIONS

Proper debonding of the test pile in the active soil wedge is essential for saving enough effective test load for the deeper soil layers where the bearing capacity will be generated in the final situation of the quay wall. The use of a flushed oversized casing driven with bentonite injection seems to be the best debonding measure, although the amount of force transferred to the casing is still variable. Measuring the local strain in the pile using optical sensors glued on the pile before pile driving has been a success. The measured skin friction in the final load step along the MV pile ranges between 250 kPa at the top of the Pleistocene sand layer towards 600 kPa near the toe of the pile, which is significantly higher than historic test results which were around 250 kPa.

7 ACKNOWLEDGEMENTS

The authors would like to thank the Port of Rotterdam Authority for the innovative mindset and belief in the benefit of the test campaign.

8 REFERENCES

Brassinga, H.E. 1987. *Proefbelasting van twee M.V.-palen aan de Brittaniëhaven te Rotterdam*, Gemeentewerken Rotterdam, Rotterdam.

de Gijt, J.G., Brassinga, H.E. 1990. Diepwaterterminal op de Maasvlakte te Rotterdam – Geotechnische aspecten zeekade en bewerkingskade, *PT civiele techniek* **2**, 7-13.

SBRCURnet, 1997. CUR 166 *damwandconstructies/sheet pile constructions, 3rd edition*, CUR, Gouda

van Paassen, B.P.H., van Dalen, J.H. 2009. High load tests on VM-piles for the quay wall Euromax. *Proceedings of the 17th International Conference on Soil Mechanics and Geotechnical Engineering* (Eds: Hamza, M. et al.), 1143-1146. IOS Press., Amsterdam.