

High load tests on VM-piles for the quay wall Euromax

Essais en charges élevées de pieux Müller pour le mur de quai Euromax

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ABSTRACT

At the Port of Rotterdam, BAM Civiel has recently built a 1900 m long quay wall which now serves at the new deep-water container terminal 'Euromax'. Anchorage of this quay wall, which is designed for 12.500 TEU ships, is provided by 56 m long VM-piles: slender H-shaped beams that are grouted during pile driving. As a result of the functional requirements of the quay wall and the center to center distance of 5.5 m, these MV piles are designed to resist a design load of 7200 kN/pile.

Because of the unusual length and cross section of the piles, 5 VM-piles have been tested to a load of 9100 kN/pile. As the predicted ultimate resistance of the piles was higher than the actual testing load, additional analyses and extrapolations were required to approve the performance of the piles.

Based on the tests, analyses of the test results and comparisons made with previous loading tests on MV piles, conclusions were drawn with respect to the applicability of the design rules used in this case. Also the applicability of creep criteria for this case has been evaluated.

RÉSUMÉ

Dans le Port de Rotterdam, BAM civil a construit récemment un mur de quai de 1900 m de long, qui sert maintenant pour le nouveau terminal à conteneurs en eaux profondes 'Euromax'. L'ancrage de ce mur de quai, équipé pour recevoir des navires de 12.500 EVP, est assuré par des pieux Müller de 56 m de long: il s'agit de poutrelles élancées en H, injectées avec du mortier durant le battage. Pour satisfaire aux spécifications fonctionnelles du mur de quai et à un entraxe de 5.5 m, ces pieux Müller ont été conçus pour pouvoir supporter une charge nominale attendue de 7200 kN/pieu.

En raison de la longueur et de la section inhabituelles de ces pieux, 5 pieux ont été testé sous une charge de 9100 kN/pieu. La résistance ultime prévue des pieux étant plus élevée que la charge effective durant l'essai, des analyses supplémentaires ainsi que des extrapolations ont été nécessaires pour valider la capacité des pieux.

A partir des essais, des analyses des résultats de ces essais ainsi que de comparaisons effectuées avec des essais de mise en charge de pieux Müller préexistants, des conclusions ont pu être tirées quant à l'applicabilité des règles de conceptions utilisées dans ce cas. L'applicabilité du critère de fluage a été également évaluée pour ce cas.

Keywords : loading tests, test piles, tension piles, MV Piles, quay wall, creep rate

1 INTRODUCTION

At the Maasvlakte area at the Port of Rotterdam, BAM Civiel has recently built a 1900 m long quay wall which serves at the new deep-water container terminal 'Euromax'.

The quay wall has a 27 m retaining height, and is designed for 12.500 TEU ships and a surface load of 40 kPa. The wall consists of diaphragm walls to a depth of NAP -34 m, with a 18 m long L-shaped relief structure on top. This relief structure is supported by two rows of compression piles; anchorage is provided by approximately. 56 m long MV-piles with a centre to centre distance of 5.5 m.

In this article the high load tests as well as the various design aspects of these MV piles are discussed.

In 2005 the Port of Rotterdam awarded the project to contractor BAM Civiel BV by means of a design and construct contract. The design has been made by BAM Infraconsult B.V., in accordance with the 'Manual Quaywalls' (CUR, 2003), edited by Rotterdam Public Works. The MV piles have been installed by BAM Grondtechniek B.V..

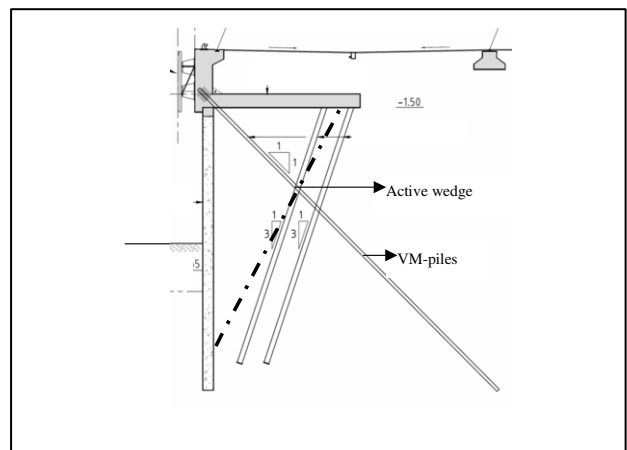


Figure 1: cross section quaywall 'Euromax'

VM-piles (Muller-Verfahren) have regularly been applied in the past at the Rotterdam harbor area because of their high bearing capacity for tension. During driving of the H-beams

grout is supplied by grout pipes leading towards reservoirs mounted at the toe of the pile. The shell of grout that is formed around the pile, is advantageous for the drivability and -after hardening- also for shaft resistance.

The VM-piles applied at Euromax, with a length of 56 m and an unusual length to width ratio of the steel profile (HE 600B) are out of the range of experience of former applications in the region. By application of a steel grade S355 a yield strength of 9500 kN is reached.

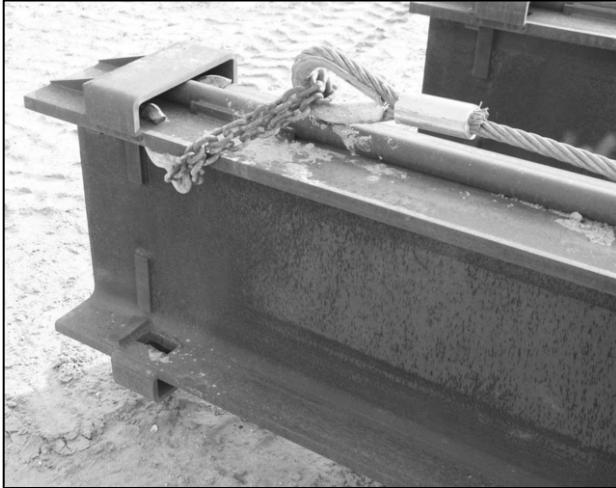


Figure 2 Beams and grout reservoirs at toe of H-beam

1.1 Ground conditions

The quay wall is built from the surface level (NAP +5 m) of the reclaimed Maasvlakte where the upper 25 m consist of sand with varying densities, and alternating silty sand and (thin) clay layers. A silty clay-layer with up to 3 m thickness is located at 27 m depth, showing cpt values of 1 - 2 MPa. Underneath the Pleistocene sand ('Kreftenheye') is found, characterized by high cpt values of up to 60 MPa indicating its very high density.

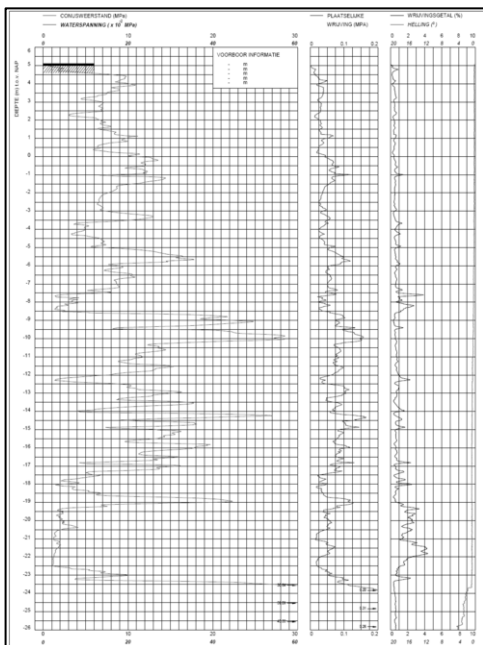


Figure 3 Typical CPT- graph

1.2 Geotechnical design of VM-piles

As a result of the functional requirements of the quay wall (crane- bollard- water- and ground load) , these MV piles have to resist a service load of 5200 kN/pile, and - taken into account the design philosophy - a design load of 7200 kN/pile.

The required pile length has been calculated according to the Dutch design rule for tension piles as described in (CUR, 2001). Here the shaft friction is defined as a function of the cpt-value q_c and the friction ratio α_t . The pile resistance follows from integration of the shaft resistance along the pile and, additionally, reductions by the required safety factors:

$$R = \int_0^L P_{rz} O \xi / \gamma_m dz \quad \{1\}$$

Where:

$$P_{rz} = \alpha_t * q_c \leq P_{rz \text{ limit}}$$

P_{rz} = shaft resistance (kPa)

α_t = 0.012, friction ratio according to (CUR, 2001)

q_c = cpt-value (kPa)

$P_{rz \text{ limit}}$ = limit shaft friction = 180 kPa

O = circumference, defined by the outline of the reservoirs near the toe (figure 2)

ξ = factor depending on the number of cpt's available, 0.85

γ_m = partial safety factor = 1.4

z

It should be noted that no shaft resistance has been taken into account above the level that corresponds with the crossing level of the active soil wedge and the VM-piles (NAP -11 m; figure 1).

2 INSTALLATION OF THE PILES

The reduction of shaft resistance during driving caused by the presence of the grout around the shaft – still in liquid state - enables the installation of MV piles to considerable lengths into the very dense sand layers. In this way the energy of the hammer is almost completely transferred to the toe of the pile. A continuous supply of grout is essential in order to make sure that the pile is surrounded by grout over the full length. Uninterrupted driving is important to prevent the hardening of the grout during the installation process.



Figure 4 Pile Installation

The great length of the piles combined with the high density of the Pleistocene sand has, nevertheless, raised questions about the drivability. Initially, based on previous experiences, a hydraulic IHC S-90 block with a ram-weight of 9 tons has been used, resulting in driving times of 2.5 hours showing peaks of up to 3 to 4 hours. Changing, during the project, to a ram of 12 tons improved the driving time considerably.

3 LOAD TESTS

As the VM-piles used at Euromax have been out of the range of experience of former applications in the region, 5 loading tests were agreed by the contractor and the harbour authorities to prove the applicability of the design rule. The site supervision, instrumentation, data collection and interpretation of the tests were carried out by Rotterdam Public Works.

The test arrangement, which was constructed by BAM Project Support B.V., consisted of a special test frame with a span of 11 m, 2 hydraulic jacks with a capacity of 500 tons, strain gauges at various levels along the pile and high resolution sensors to measure the pile displacements. In order to minimize friction along the upper layers, this part was injected with bentonite instead of grout.

At these tests a load of 9100 kN, equal to 95% of the yield force, was gradually applied at a rate in accordance with the Dutch guideline for sheet piles (CUR, 1993). However, as a result of the safety approach, the actual – unfactored – geotechnical bearing capacity that was to be approved was $7200 * 1.4/0.85 = 11850$ kN, exceeding the tension capacity of the steel. Testing could therefore not be done up to the expected pull out force, additional analyses and extrapolations were necessary to approve the performance of the piles (chapter 4).



Figure 5 Test frame

4 INTERPRETATION

4.1 Extrapolation to ULS

The pile is supposed to have reached its ultimate limit state at the point where the displacement of the head increases continuously, under constant loading conditions. This has not happened for any of the test piles. To predict the ultimate load capacity, an extrapolation of the load-displacement curve has been made. Several authors (Chin, 1970; van der Veen, 1953) have described generally accepted methods of extrapolation, but these methods are meant for compression piles, and are not applicable for tension piles. For previous applications of tension piles, Rotterdam Public Works have derived that better results can be achieved by using a method based on the displacement that remains after removing the load at each loading step (Opstal et al., 1993). This method is based on the following formula:

$$w(F) = w(F) * F / F_u + C \quad \{2\}$$

with:

$w(F)$ = remaining displacement after removal of the load F
 C = constant
 F_u = ultimate load

As an example a graph is presented in figure 6, showing the results of test pile 7. For each loading step the w is plotted against $w * F$. The tangent of the angle of the best-fit line through the points per loading step is equal to $1/F_u$.

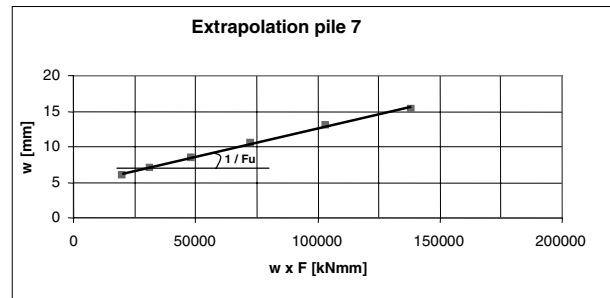


Figure 6 Example of extrapolation method

4.2 Normal force and shaft friction

To calculate the friction between piles and surrounding sand the normal forces along the pile shaft have been determined by interpreting the results of the strain gauges that were placed at several depths along the pile. Because the strain gauges were applied after installation of the piles, it was not possible to directly calibrate the relation between measured strains and normal force.

In this case it appeared that there was no pure linear relationship between stress and strain: Hooke's law did not apply. The explanation for this phenomenon lies in the dimensions of the pile in this case. The difference in thickness between flanges and body provokes differences in cooling down speed during production, which causes initial "frozen" stresses in the beam. Because of those stresses, even at the lower test loads, local yielding occurs during the test. This phenomenon causes strains, larger then would be expected from Hooke's law.

To take this effect into account, the relation between measured strain and normal force has been determined for each test pile, using the strain-gauges located on the pile head. Since the normal force adapted to the pile is equal to the normal force at the level of those upper strain-gauges, the relation between strain and normal force could be described as

$$F_n = a * \epsilon^2 + b * \epsilon \quad \{3\}$$

The normal force at deeper levels in the pile was calculated by using the same formula, with constants a and b as found for the pile head. The given relation was used for primary loading situations. For reloading the relation between normal force and strains was fully in accordance to Hooke's law, based on a constant modulus of elasticity of the steel.

The most reliable result for the normal force was found for pile 7; the normal force for this pile is shown in figure 7 for the several load-steps that were applied.

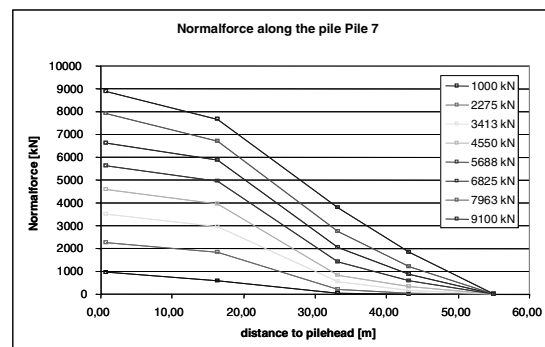


Figure 7 Normal forces derived from strain gauges

From the difference in normal forces between two levels, the average friction force along the pile shaft can be calculated for each load step by dividing the total force per layer by the thickness of each layer. The result is plotted in figure 8.

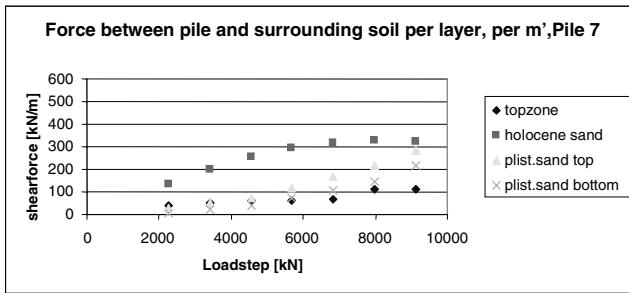


Figure 8 Average friction per layer

From these forces the friction-ratio α_t can be calculated. A value of 1,6% is found for the higher (Holocene) sand layers. For the deeper (Pleistocene) sand layers the ultimate skin-friction is not yet reached at the highest load step, as the friction force is still increasing at the last load step. The maximum value for α_t reached is 1,15%, if q_c is considered limited to 15 MPa, as described in (CUR, 2001).

4.3 Creep-behaviour

During each load-step the creep behaviour of the piles is examined by calculating the creep rate k :

$$k = (u_2 - u_1) / \log(t_2/t_1) < 2 \text{ mm} \quad \{4\}$$

with

$k =$ creep rate [mm]

$u_2, u_1 =$ displacement of the pile-head at time = t_2, t_1 respectively

In accordance with (CUR, 1993) the maximum allowable creep rate k was 2 mm during each load-step. From experience with earlier projects in the Rotterdam Harbour, the creep criterion has never been an issue for VM-piles before. In this case however, relatively large values for k varying from 1 to 2,2 mm were measured.

If the value for k is high this means that extra attention is necessary for two reasons:

1. The deformation in time can get too large
2. The fact that the deformation in time has not stopped can be an indication of the initiation of progressive collapse.

From calculation it was proven that the first item is no issue in this case; the total deformation at the end of the lifespan would still be acceptable if the creep rate k would, on a logarithmic scale, stay constant during its lifespan.

To be sure about the second item, the long-term behaviour of the bearing capacity, the last load step for pile 55 (the pile for which the highest value for k was found), was prolonged from 1 to 4 hours. After this period it was concluded however that the creep k had not diminished yet upon which a second test has been performed, three days after the first test. In this second test the same maximum load was kept constant for a period of 8 more hours on the same pile. Finally, after 4,5 hours the pile displacement came to a stop with creep rate $k=0$ (figure 9).

A logical explanation for the difference in behaviour of the MV-piles in this project, compared to the previous applications, can be found in the bigger length of these piles and the very high steel stresses. This results in a larger extension of the piles, causing the front part of the pile to undergo a larger shear-deformation versus the surrounding sand, before the skin friction along the bottom part of the pile can be developed.

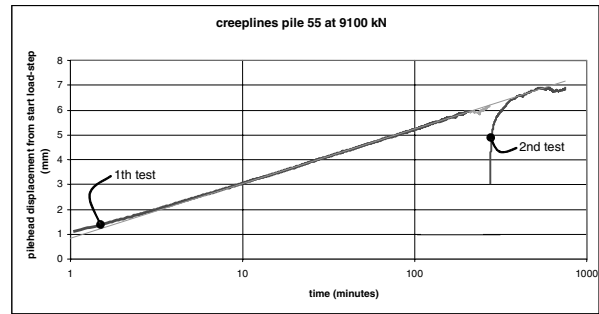


Figure 9 Creep rate of pile 55

Creep may indicate the risk of a loss of bearing capacity along the upper part of the pile, as a result of the large shear deformation. By prolonging the test it was shown that the creep eventually stopped, which proves that the deeper part of the pile is able to compensate for the loss of bearing capacity of the upper part of the pile.

At last, it should be noted that in this case this criterion {4} is disputable, due to the fact that a significant part of the maximum load is only present for a very limited time. This aspect has however not been taken into account and the total maximum load was considered to be long-term present.

5 CONCLUSIONS

The geotechnical bearing capacity of the VM-piles at Euromax has been approved by high load tests in combination with the extrapolation methods that have been derived at previous projects [Opstal et. al., 1993], and strain gauges measurements

From strain gauge measurements, a friction-ratio α_t of 1,6% was found for the higher (Holocene) sand layers. For the deeper (Pleistocene) sand layers a maximum value for α_t of 1,15% was reached, although, at the highest load step, the ultimate skin friction was not reached yet.

The tested piles were subject to more creep than observed at previous projects with VM-piles in the Rotterdam Harbour. This is likely due to the relative large elastic extension of the piles, caused by both the great length of the piles (56 m) and the relatively high stresses (almost equal to the yield stress).

For this project, the measured creep of the piles was not more critical than the ultimate geotechnical bearing capacity.

The design rule used for this project, based on $\alpha_t = 1,2\%$ and q_c limited to 15 MPa has resulted in a safe design.

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