Dynamic analysis of large diameter piles Statnamic load test

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ABSTRACT: In order to check the bearing capacity of a newly introduced large diameter casing pile, both CPT analysis and static load testing was applied. In addition to that, two piles where tested using the Statnamic load testing technique. With respect to the interpretation, in addition to the standard Unloading Point method by Middendorp et al. (1992), a dynamic analysis with a dynamic finite element code, i.e. Plaxis, was done; further the static load test results were used to calibrate the soil parameters for the analysis. Comparing the Dynamic load testing results according to Middendorp with the Numerical results; it came forward that some additional mass below the pile tip, more or less moving with the pile, needs to be taken into account. Further after Mullins et al. (2002), the damping is calculated for the stationary part of track 4. Overall this gives a better agreement with numerical analysis for the bearing capacity. Based on an extrapolation of the physical test up to deformation required by NEN 6743-1 (2006), an ultimate load bearing capacity of 15.5MN was established.

1 INTRODUCTION

Related to the city extension Leidsche Rijn near Utrecht, The Netherlands, and the extension of the railway between Utrecht and Gouda to four railway tracks, a total of 20 new viaducts need to be built. In order to improve the foundation concept of the classic multi pile pier foundation, an alternative was proposed that includes the use of bored casing piles with a diameter of 1.65m. This innovation reduces the original design of 48 prefab concrete piles to 6 large diameter casing piles only.

Since the bridges are intended to support the railway track, the design requirements are very strict. On the one hand the piles should be capable to bear a design load of at the least 12,000 kN, on the other hand the deformation during train passing’s should be less than 10 mm. Since bored piles show a relatively less favourable load-displacement curve compared to driven piles, to improve the stiffness, the introduction of pile tip grouting device adopted.

Both the large diameter casing pile in combination with the grout injection device at the tip was a first time application in the Netherlands. For the evaluation of the bearing capacity, as a start CPT’s after prestressing the grouting device, were taken and evaluated as a first step in checking the influence of the installation procedure. In addition a total of three piles were instrumented and tested on site by means of a static load test and further the deformation of all piles were monitored during construction and the actual passing of the first trains in December 2007. Details are given in de Jong et al. (2010). In addition, after that it became clear that a suitable testing device would be available; two

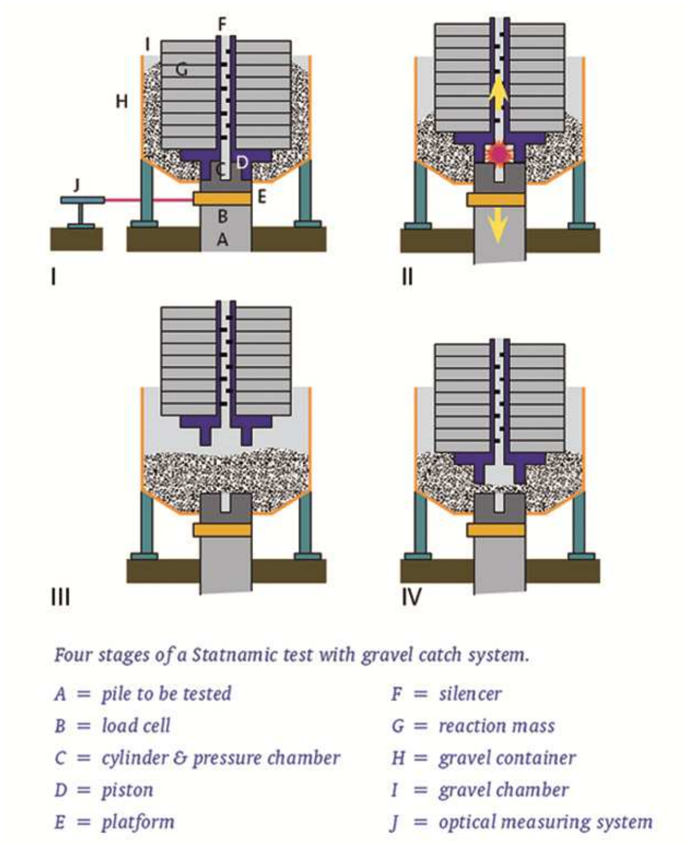


Figure 1 Stages in statnamic testing

additional test piles were tested with a 16MN statnamic device.

In addition to the standard procedure of checking these tests using the UPM (Unloading Point Method), by Middendorp et al. (1992), a dynamic axisymmetric Finite Element analysis of the load tests was performed. The result of the static load tests both with respect to load and deformation was used to calibrate the soil parameters for the dynamic numerical analysis. Further the dynamic response as calculated with Plaxis was put into the same UPM evaluation procedure for further corroboration of the method.

In this paper a comparison between the UPM evaluation procedure by Middendorp and the Numerical analysis with Plaxis will be described. Attention shall be given to the effect of drained bearing capacity for design and undrained behaviour during the test.

Some typical modelling issues such as mesh finesse at pile tip and dynamic effects in interface elements will be discussed.

In addition to understanding the dynamic behaviour of the pile, the calibrated numerical model opens up the possibility to evaluate the equivalent static load bearing capacity, which can be compared to the Dutch code for static load capacity NEN 6743 (2006).

2 STATIC LOAD TESTING AND SOIL PARAMETER ESTIMATION

Before it was decided to do statnamic testing a static pile test was done; although it was known in advance that the limit load with respect to bearing capacity could not be reached.

At bridge number 6 a row of three, 17.85 m long, 1.65 m diameter piles at a centre to centre distance of 3.75 m where individually tested. The goal of these tests was verification of the load displacement behaviour up to 8,000 kN, i.e. the pile stiffness.

Strains were measured at three levels with vibrating wire strain gauges, and a load cell was installed at the top. Further displacements were measured at the pile head by high accuracy levelling. In addition to the deadweight of 1,000kN the reaction frame was anchored to the ground by 8 Gewi-anchors with a capacity of 1,250kN each.

Prior to pile installation one CPT was carried out at the centre of each pile. Based on the 3 CPT's before pile installation the ultimate bearing capacity of the piles was estimated to be about 20,000kN.

During the static test, the piles were loaded in steps of 1,000kN. The load was then kept constant for 1 hour, a time that was extended if the deformation rate was above 0.3mm/h up to a maximum of 4 hours. At a load of 6,000kN the piles were unloaded to 4,000kN and reloaded to 6,000kN in steps of 1,000kN. Only one of the piles was loaded up to 8,000kN. More details and results of the Static Pile test can be found in de Jong et al. (2010).

After that the Static load tests were done, the introduction of a 16MN Statnamic device in the Netherlands by Fugro made it possible to test the piles to loads that would approach the ultimate bearing capacity of the piles.

In addition to the field test it was decided to try for a dynamic finite element analysis to get a better understanding and to create added value to the field test.

The Statnamic load test were done not too far from the site were the Static tests were done, and had a similar soil layering. Therefore the static tests were used as an additional source of information for calibration of the soil parameters for the dynamic analysis. The static analysis was back-analysed and soil parameters were calibrated and applied for the first prediction of the Statnamic test.

3 STATNAMIC TESTING

In November 2008 two Statnamic load tests were carried out on sacrificial test piles that had been installed in 2005. Originally it was intended to test these piles to refusal by means of a static load test. Such a test would have required a load of approximately 20,000kN, even though the piles have a limited length of 13m.

In a Statnamic load test, pile loading is achieved by launching a reaction mass. Due to the high acceleration of the reaction mass, the effectiveness of the load is increased by a factor of about twenty. The loading is perfectly axial, the pile and the soil are compressed as a single unit and the static load-displacement behaviour of the pile can be determined if a series of tests is performed.

Given the fact that the information of the static load test up to a load of 8,000kN was already available, the two test piles were only tested once with the maximum Statnamic load of approximately 16MN. The general description of the statnamic test has been given by Middendorp et al. (1992). The test procedure itself is according to the draft European guideline (Hölsher and van Tol, 2009).

4 EMPIRICAL EVALUATION OF BEARING CAPACITIY

According to Middendorp's (1992) simplified evaluation procedure, that dynamic force balance may be written as:

$$F_{stat}(t) = F_u(t) + F_v(t) + F_a(t) \qquad (1)$$

Where

$F_u(t)$ = static soil reaction (point and shaft)
$F_v(t)$ = damping force (depends on pile velocity)
$F_a(t)$ = inertia force, depending on mass and acceleration

Both $F_{stat}(t)$ and $F_a(t)$ have been measured or can be inferred from the measuring data. Whereas $F_u(t)$ is the unknown soil resistance we want to establish and further $F_v(t)$ is also relatively unknown and depends on the known pile velocity.

However, as the loading time is relatively long compared to the velocity of pressure waves in the pile, which is in the order of 3,800m/s, whereas the loading time is in the order of 0,08s, pressure waves may travel up and down the pile, 12 times within this period. For that reason the load may be judged to act semi-static. The effect will be that the pile will displace more or less as a rigid mass. For that reason it is assumed that the time of maximum displacement equals zero velocity of the pile. Realising that at this point; i.e. referred to as the "Unloading Point" the damping is zero, the pile resistance can be calculated as:

$$F_u(t_{u\max}) = F_{stat}(t_{u\max}) - m.a(t_{u\max}) \qquad (2)$$

According to the UP method it is assumed that the soil resistance between maximum statnamic force and standstill, has developed beyond the point of maximum bearing capacity commonly referred to as trajectory 4, is nearly a constant. If this is really the case may be disputed. However, if this is assumed equation 1 may be rewritten as

$$F_{stat}(t) = F_u(t_{u\max}) + C_4 \cdot v(t) + m \cdot a(t) \qquad (3)$$

Where
C_4 = damping coefficient
m = Pile mass
$a(t)$ = Pile acceleration (measured)

Given that Statnamic loading as well as the pile accelerations are measured, the average damping factor can be calculated according to:

$$C_4 = \frac{1}{n}\sum_{j=1}^{n}\frac{F_{statn}(t_j) - F_y - m \cdot a(t_j)}{v(t_j)} \qquad (4)$$

With

$$F_y = F_{statn}(t_{u\max}) - m \cdot a(t_{u\max}) \qquad (5)$$

With the damping coefficient determined, the soil resistance can be back-calculated for the whole track between maximum statnamic loading and maximum displacement.

It is customary to back analyse the damping for this whole trajectory and taking the average value. According to this standard interpretation method, the Soil resistance can be back analysed and will result

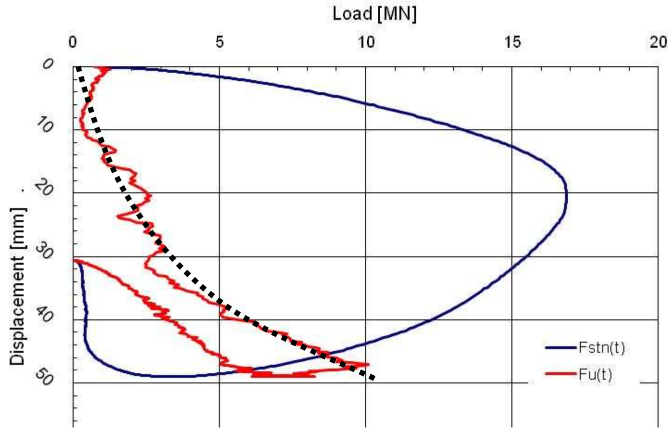


Figure 2 Soil resistance as back-analyzed from the Statnamic test, assuming averaged damping in track 4 of the test; concave loading curve is found

in the curve given in Figure 2. Contrary to expectation, the curve has a concave form; moves upward whereas static loading tests normally show a more convex; i.e. hyperbolic shape.

Evaluating this phenomenon in more detail the result seems to be negatively influenced by the averaging procedure for the damping ratio. According to Mullins et al. (2002), a better result can be found by taking the static value of the damping ratio as indicated in Fig. 3, instead of averaging the damping in the whole trajectory. With this adaptation the result is improved, corroborates better with the numerical results and shows the common observed hyperbolic shape, see also Fig. 10 with the improved curve.

Based on the evaluation with the UP method, it was established that the largest soil resistance was reached for a deformation of 47 mm, which is further taken into account.

Before going into more detail into the final results of the Statnamic test, some details of the numerical analysis will be given, that shed some more light on the way the test results may be interpreted.

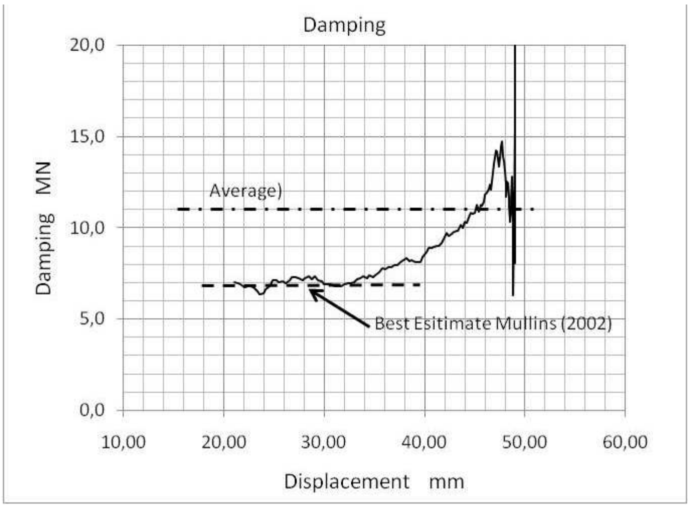


Figure 3 Interpretation of Damping coefficient C_4, interpretation after Mullins et al.2002

Table 1 Soil parameters after calibration with static loading test

Soil type	Level NAP	γ_{dry}	γ_{wet}	E_{50}^{ref}	E_{ur}^{ref}	m	$\gamma_{0,7}$	G_0	C	ϕ'	ψ	K_0
	[m]	kg/m^3	kg/m^3	kN/m^2	kN/m^2	[-]	[-]	kN/m^2	kN/m^2	°	°	[-]
Holocene (loose) sand	+0.50 - - 7.10	18	19	10000	30000	0.7	10^{-4}	37500	1.00	28	0	0.50
Pleistocene (dense) sand	-7.10 - down	18	19	50000	150000	0.7	10^{-4}	187500	0.03	38	8	0.75

m = power in hyperbolic relation between elastic stiffness and isotropic stress; e.g. 0,5 for sand and 1.0 for clay
$\gamma_{0,7}$ = threshold for small strains in the Hs_{small} model; i.e. the strain at which the shear modulus has reduced to 70 % of G_{max}
G_0 = G_{max} = shear modulus at very small strain; may be compared to the shear modulus in dynamic analyses

5 NUMERICAL MODELLING OF STATNAMIC LOAD TEST

For the numerical analysis of the load test an axisymmetrical model was made using dynamics module of the Plaxis 2D V9 finite element code.

With respect to the numerical analysis distinction must be made between the prediction for the first statnamic test and back-analysis on the first and the second test.

For the prediction of the first statnamic test a relatively detailed soil layering was applied, based on a direct interpretation of the CPT data in combination with table 1 of NEN 6740 (2006). Further the parameters were calibrated with respect to the static load test, see section 2, carried out in 2005, recognizing that both the piles in the static tests (with a length of 18m) and those in the statnamic tests (with a length of 13m), were positioned in the same geologic layering.

Further a first analysis of the static bearing capacity of the pile indicated the stiffness effect of the grout bag at the pile toe. From measurements during installation and grouting of the pile, it was known that a pile rise of about 8 mm was observed. In the finite element analysis the effect was modelled as a volume strain in a zone of 17 % in a zone of 0.5 m below the pile tip, which gave the same rising of the pile. A first indication of the Static bearing capacity of the pile, with and without grouting at the pile toe is indicated in Fig. 4.

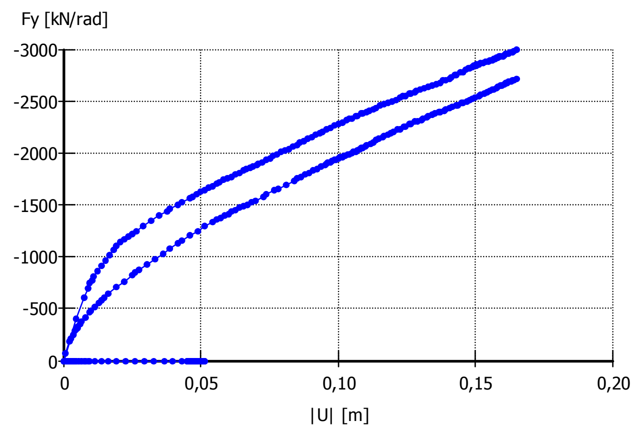


Figure 4 Simulation of Static analysis of the test pile configuration (13 m pile) with Plaxis; with (upper curve), or without base grouting of the pile toe (lower curve), (Plaxis output loading per radial).

The static model test has been performed up to a total settlement of the pile head of 0.1 D (0.165 m) according to the NEN 6743 (2006). The load capacity based on the static numerical analysis indicates a bearing capacity of:

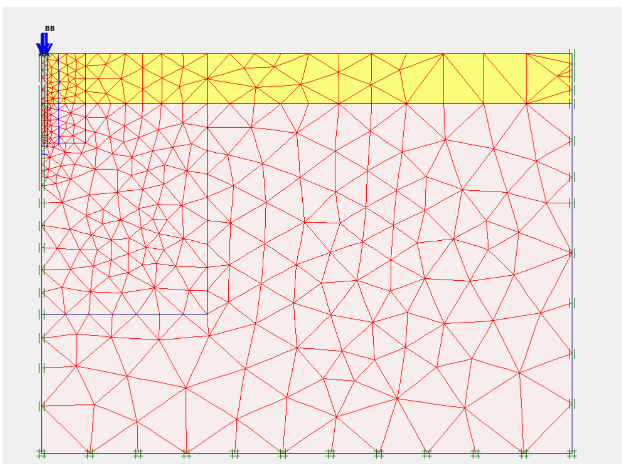
$$F_{,d} = 2\pi \cdot 2.988 = 18.78 \text{ MN} \quad (6)$$

Please note that the output of Plaxis, Fig. 4, presents the load per radial, so this result needs to be multiplied with 2π, to get the full bearing capacity of the pile.

With respect to the dynamic prediction for the first Statnamic test, the displacements where over-estimated to be between 130 and 180mm, whereas the actual displacement was about 50mm.

Given this observation, and in order to improve the model the tested data was back-analyzed, to reduce the differences between numerical and field test. For that, a sensitivity analysis with the model, and the applied parameters was performed:

- A first observation in this analysis was that the actual loading time during the test was a little shorter than assumed in the prediction, and subsequently contained a little less energy.
- Secondly back-analysis indicated that the friction angle, of the Pleistocene sand layer at the pile toe, must have been higher than first assumed, i.e. a better match was found for a friction angle $\phi = 38°$ instead of 35°.



5 Finite Element model for dynamic load test.

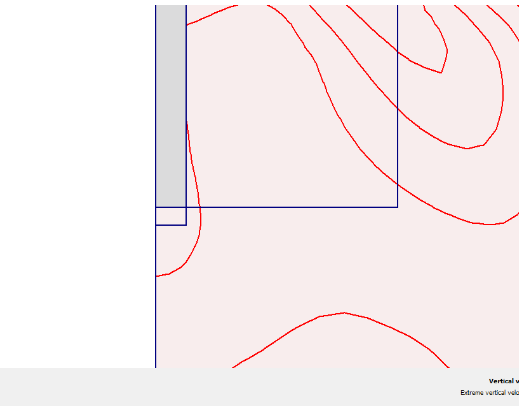


Figure 6 Characteristic displacement field in Plaxis dynamic analysis, indicating that a zone underneath the pile toe moves more or less with the pile.

- Subsequently the dilatancy angle was adjusted, assuming the common relation $\psi = \phi - 30^o$, and was increased up to $\psi = 8^o$ for this layer.
- Introduction of the Hssmal, small strain material model, see Benz (2007), improved the results; further the parameters for this material model were evaluated, indicating that with $\gamma_{0,7} = 10^{-4}$ the best agreement was found.
- In addition to that, the damping factors where slightly increased up to Rayleigh $\alpha = 10^{-3}$ and Rayleigh $\beta = 1.75 \times 10^{-3}$, see Zienkiewicz et al. (1991).

In addition to that, the mesh needed to be refined around the pile tip, further some variation in Holocene upper layers was ignored as some of the very soft layers near the soil surface seemed to destabilize the numerical analysis. Due to tensile stresses at the pile delaminating of the interface elements seemed to develop leading to numerical instability. As a solution averaged soil parameters for the Holocene layer were assumed which seemed not to affect the overall result to much, see Fig. 5 and table 1.

5.1 *Drained and undrained analysis*

For the static analysis as indicated in Fig 4 drained analysis was assumed. For the statnamic test, given the large diameter and short duration of the test, only 0,08 sec for the test, undrained behaviour must be assumed. The effect is illustrated in Fig 7, where the results of both a drained and an undrained analysis are given. Due to undrained behaviour the bearing capacity at the toe will be limited due to excess water pressure, whereas due to dilation of the soil at the pile shaft the friction in undrained analysis leads to a higher soil resistance. Fig 7 indicates that undrained behaviour at the shaft friction is dominant.

For the interpretation of the dynamic analysis the ratio between the two curves will be taken into account.

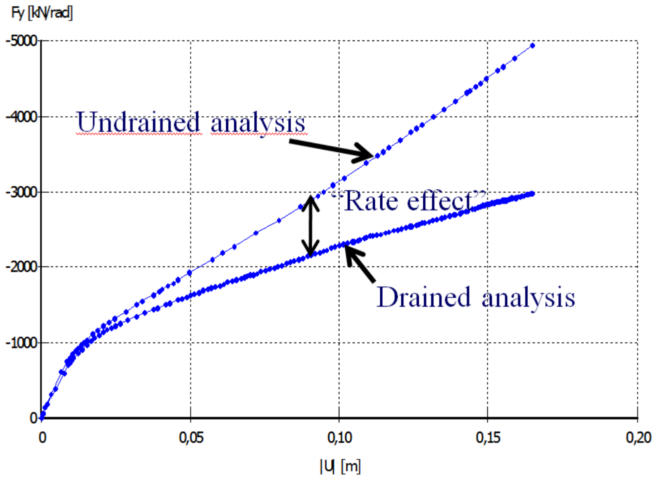


Figure 7 Static analysis with the optimized Finite element model; drained and undrained

5.2 *Dynamic Analysis with Plaxis*

Using the same procedure as for the direct interpretation of the Statnamic test, the bearing capacity was back-analysed from the numerical analysis, i.e. the UPM method as explained in the previous paragraph. Based on several variations of the numerical analysis, and considering that the elements in the numerical model itself do not differentiate between pile elements and soil elements, it became clear that to explain the test properly it is necessary to account for some moving soil mass underneath the pile toe. Here, to be conservative, a soil volume of 1 times the diameter and 0.6 times this depth was adopted. To get a full agreement with the static analysis the size of this soil mass needed to be increased up to 2 times the pile diameter, which in itself is not unlikely, see Fig 6, but was not applied further, in order to have a conservative result.

Compared to more slender piles, the effect of soil mass moving with the pile tip the effect here was more dominant due to the relative large diameter of the pile. The result of the back analysis is indicated in Fig. 8. and Fig. 9.

In section 5.1 it was argued why the dynamic analysis was performed undrained whereas for the

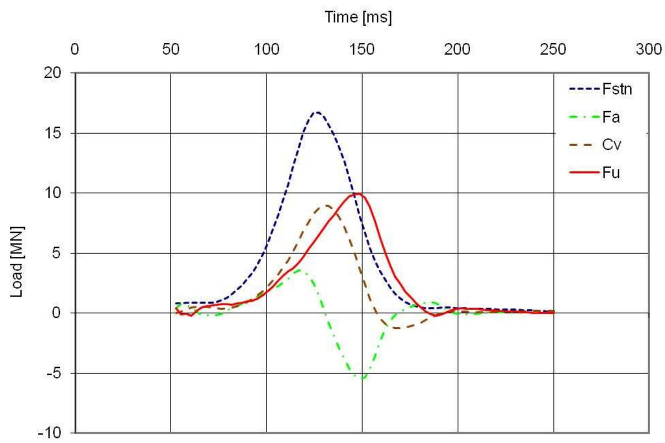


Figure 8 Results of the dynamic deformation analysis back-analyzed with Plaxis

bearing capacity it is customary to assume drained soil behaviour. The difference between drained and undrained behaviour is partly contributing to what is known as the rate-effect.

Here this effect is discounted for by taking the ratio between the drained and the undrained analysis as presented in Fig. 7, for the measured deformation of 47mm; which gave a reduction factor of 0.84,

In Fig. 9, the characteristic load displacement curve based on the back-analysis with Plaxis is given, that may be compared to Fig. 10. The agreement overall is reasonable, the only shortcoming is that it seems to be difficult to represent the elastic unloading at the end, that lags behind in the numerical analysis. This test result in itself may however also be disputed, there is reason to assume that the pile tip has come loose in rebound.

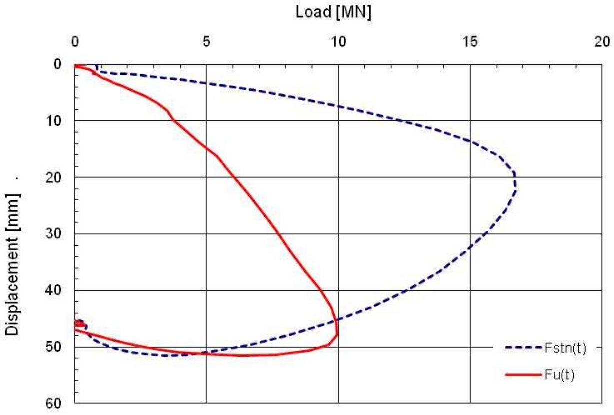


Figure 9 Results of Numerical simulation of the Statnamic load test on 17 nov. 2008 evaluated with the UP method. To compare with the physical result as indicated in Figure 10

Based on the UPM evaluation of the numerical analysis a maximum pile load during statnamic testing was found of: $F_{r,I}$ (δ = 47 mm) = 9.84 MN

In order to compare this load with the ultimate bearing capacity according to a CPT or a static load test, it is necessary to consider not only the drainage effect, but also the fact that the deformations derived with the Statnamic test do not satisfy the necessary deformation for a static load test, that requires a displacement of 0.1 D_{eq}, or in this case 0.165m.

Referring to the load development curves in Fig. 7, an exponential extrapolation is assumed. Further, correcting for the rate effect as indicated in Fig. 7, the bearing capacity of the pile is approximated as:

$$F_{r,0,1D_{eq}} = 0.84 \cdot 9{,}84\sqrt{\frac{0.165}{0.047}} = 15.49 \text{ MN} \tag{7}$$

Comparing this value with the results of direct static analysis it is concluded that:

$$15.49 \text{ MN} < F_{0,1Deq} <= 18.78 \text{ MN} \tag{8}$$

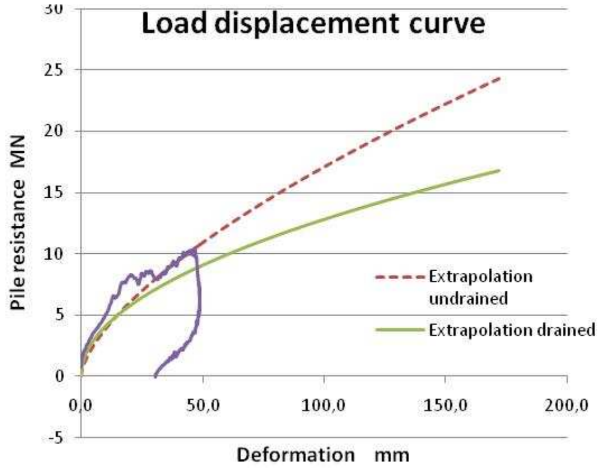


Figure 10 Hyperbolic extrapolation of the load displacement curve acc to eq. no 8

6 CONCLUDING REMARKS

- A dynamic analysis of the statnamic pile test helps to explain for the deformation behaviour.
- In order to prevent delaminating of interface element at the pile in the softer upper layers it was deemed necessary to apply averaged soil parameters for the whole upper layer.
- Rate effects seems to be partially explained by undrained behaviour of the soils and partially by soil mass moving with the pile that needs to be taken into account in order to get agreement between numerical and physical test.
- In order to get a reasonable corroboration between test and back-analysis one needs to realise that for the latter mean values of soil parameters are needed whereas for design it is customary to use characteristic i.e. conservative values.
- Overall the agreement between field test and numerical model seems to be reasonably good.

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