The influence of pore pressures on penetration forces in sand and clay

L'influence des pressions de pore sur les forces de pénétration dans du sable et de l'argile

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ABSTRACT
The deformation behaviour of soil is rate dependent. In sand, a higher deformation rate generally leads to excess pore water pressures, which influence stiffness and strength. For normally consolidated clay, a higher deformation rate will lead to lower forces because undrained behaviour occurs at a higher deformation rate. This paper describes two experiments that study the rate effects: the penetration of a pile in sand, and the movement of a T-bar in clay. The pile tests were performed in a geotechnical centrifuge, the T-bar test was performed in a 1-g environment in normally consolidated clay. In both tests, the penetration rate was varied to obtain drained and undrained behaviour. Pore pressures were measured in the sand during the centrifuge test. A short description of the tests and the results is presented. A consequence is that a rapid pile loading test may over-predict the static bearing capacity of a pile in sand, while very slow movements in clay (for example during settlement) may lead to higher penetration forces than are measured in undrained tests.

RÉSUMÉ
La déformation d'un sol dépend du taux de déformation. Pour un sable relativement dense, un taux de déformation plus élevé mènera généralement à un excès de pression de pore qui influence la déformabilité et la résistance du sol. Pour une argile normalement consolidée un taux de déformation plus élevé mènera à un abaissement des forces, parce qu'un comportement non drainé se produira. Deux expériences sont décrites : la pénétration d'un pieu dans le sable et le mouvement d'une barre en forme de T dans de l'argile normalement consolidée. Les essais de pieu ont été réalisés dans une centrifugeuse géotechnique, l'essai de T-barre dans un environnement à 1 g. Dans les deux essais, le taux de pénétration a été varié pour obtenir un comportement drainé et non drainé. La pression de pore a été mesurée dans le sable pendant l'essai en centrifugeuse. Le papier décrit brièvement les essais et présente les résultats. Il est montré qu'un essai de chargement de pieu rapide peut surestimér la portance d'un pieu dans le sable, alors que les mouvements très lents dans l'argile, lors de tassement par exemple, peuvent mener à une résistance à la pénétration plus élevée que celle mesurée dans les essais non drainés.

Keywords : Centrifuge tests, element tests, rate dependency, drained behaviour, undrained behaviour.

1 INTRODUCTION

The rate of deformation plays an important role in many problems in the field of soil mechanics. Well-known examples include the dredging industry, earthquake engineering, and pile driving. However, the influence of loading rate on the stiffness and strength of soil is still only partly understood. This paper describes experimental work on two examples where the deformation rate is of importance: rapid load testing on piles in sand, and loading on sheet pile anchors in clay due to soil settlement perpendicular to the anchor rod after installation of the sheet pile. In the case of rapid loading on a pile in sand, dilatancy of the sand may lead to over-prediction of the bearing capacity derived from a rapid test. For the sheet pile anchors, the rate dependency that could be expected was not clear beforehand. The deformation rate of the settling soil can be sufficiently low that viscous behaviour of the clay may lead to lower loading on the anchor. At low deformation velocities, it is also possible that drained behaviour may lead to higher loading because the effective stresses increase.

The paper briefly describes the model tests that were performed for both situations and presents conclusions for these two loading conditions.

2 RAPID PILE TEST ON PILES IN SATURATED SAND

2.1 *Prototype situation*

The most accurate way to test the bearing capacity of a pile is to perform a static loading test. Loading on the pile is increased in steps until excessive settlement occurs. This provides an accurate prediction of the bearing capacity. Important drawbacks of this test method are the high costs and the structure required to apply loading on the pile.

The rapid load test was developed as a cheaper alternative to the static load test, and to allow testing on piles with a higher capacity. The most common rapid test is currently the Statnamic test. During this test, a weight is accelerated using combustion in a combustion chamber. The combustion chamber is located between the weight and the test pile. The inertia of the accelerated weight is used as reaction mass for a short period. Other systems using a falling mass on soft springs are also available.

In practical cases, a pile can be tested in a rapid load test using a weight that is only 1/20 of the weight necessary in a static load test. During a rapid load test, the loading duration is sufficiently long that the influence of waves through the pile can be ignored (contrary to a dynamic load test). It therefore requires less interpretation than a dynamic load test. The relatively long loading time prevents tension forces in the pile, and this means the test is also applicable for cast in-situ piles. If the rapid load test is to be used as an alternative for the static

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load test it is necessary to know the extent to which the results of both tests are comparable. The influence of loading rate effects must be considered. Soil under rapid loading will behave differently than under static loading (*e.g.* Dayal & Allen, 1975). Also, pore water pressures may develop in the sand during testing (Hölscher, 1995).

Together with a group of international experts, the Delft Cluster project "Rapid Load Test" has developed a draft European standard for executing the test. This is under discussion in CEN/TC 341, Working group 4. A guideline has now been developed for the interpretation of a rapid test. For piles in sand, a theoretically-derived correction factor for the loading rate has been proposed (Huy, 2008). Centrifuge tests have been carried out to validate the factor.

2.2 *Centrifuge tests*

Set-up

Centrifuge tests were performed to test the influence of the deformation rate on the maximum pile capacity. Centrifuge tests were chosen as the stress situation is comparable to that in the prototype and the conditions are reproducible.. The test set-up is shown in Figure 1. The sand container is filled with saturated Baskarp sand, a sand with a D50 of 135 m.

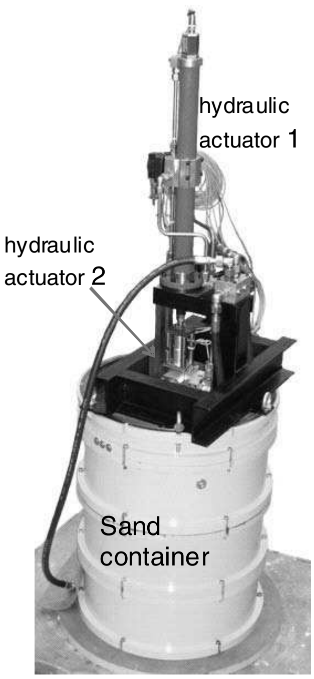

Figure 1. Test set-up for rapid pile tests in a geotechnical centrifuge.

The tests were run at 40 g. The pore water was replaced by a viscous fluid in most of the tests. This is a common procedure to achieve proper scaling of consolidation in the tests. The viscosity is normally increased linearly with the g-level, which means that the viscosity should be 40 times the viscosity of water for a 40-g test. However, the increase in viscosity in these tests was more than 40 so that the influence of drainage conditions could be tested. The last test was carried out using water as the pore fluid, in order to test a more drained situation. The viscosity increase in these tests is presented in Table 1. The viscous fluid was created by adding sodium carboxy Methyl Cellulose to the pore water, a material from the food industry.

A loading system with two hydraulic actuators in series has been developed to install the pile in the sand and to achieve subsequent rapid loading (with a duration of 6 ms). The first and largest hydraulic actuator is fixed onto the loading frame. It is used to install the pile (in flight) to its starting point prior to the static and rapid pile loading programme. The second and smallest hydraulic actuator is a fast loading hydraulic actuator with a stroke of 50 mm, which is fixed to the hydraulic actuator rod of the first hydraulic actuator. This second hydraulic actuator is used to perform the static and rapid pile loading programme. The instrumented pile is attached to the second hydraulic actuator.

The pile has a length of 300 mm and a diameter 11.3 mm. A force transducer is placed on the pile tip to measure tip resistance during the tests. The pile tip was also instrumented with a pore pressure transducer. For this purpose, small 0.5 mm holes were made in the pile tip and a filter stone and pore pressure gauge were placed in the pile.

Test performed

Four tests were performed. An initial pilot test was carried out using a different set-up and will not be discussed here. The conditions for the other tests are shown in Table 1.

In each test, the pile was installed with actuator 1 pushing the tip 22.6 cm below the sand surface. Three sets of four rapid loadings were then performed where the penetration velocities and maximum displacement differed. Once installation and each set of rapid tests had been completed, a static loading test was performed. The three sets had a maximum pile velocity of around 30, 80 and 300 mm/s. In each set, the maximum displacement was 1, 2, 5 and 10% of the pile diameter. All loadings were displacement controlled. Each rapid loading stopped when the pile was lifted with 1% of its diameter (0.1 mm) to include the unloading curve in the rapid load test. Before each rapid loading, the load on the pile head was brought back to 0 kN by manually adjusting the hydraulic actuator.

The total pile tip displacement after these tests is less than 10 mm, so all penetrations are more or less comparable. More details of the set-up are described by Huy (2008). Forces were measured at the tip and at the top of the pile, and pore pressures were measured at different locations in the sand. Only the pore pressures at the pile tip are discussed here.

Table 1. Overview of tests

Parameter	test 2	test 3	test 4
Relative density	54 %	36%	65%
Pore fluid	viscous fluid	viscous fluid	water
Viscosity (cp)	265	292	1

Results

The results of the tests show an increase in the penetration resistance as penetration velocity increases, see Figure 2.

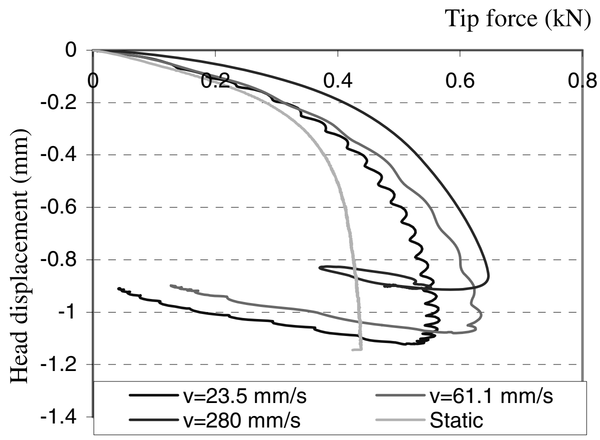

Figure 2. Force displacement diagram for different penetration velocities, Test 3.

The pore fluid plays an important role in the increase of resistance with velocity. Figure 3 shows the rapid ultimate capacity normalized on the static bearing capacity (in order to compensate for the differences in initial density between the three tests) as a function of loading velocity. The results of the test using water as the pore fluid (test 4) show almost no velocity dependency; the tests with the viscous fluid as pore fluid (test 2 and test 3) show a strong velocity dependency.

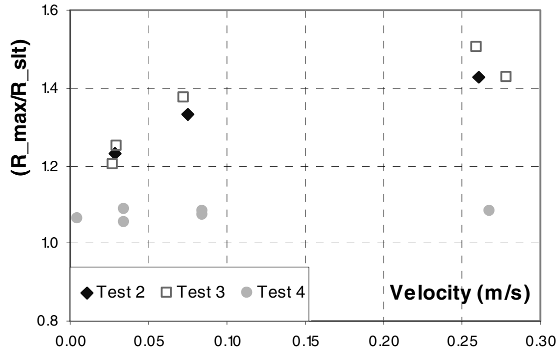

Figure 3. Comparison of rapid and static ultimate capacity for cases with viscous pore fluid (test 2 and 3) and with water (test 4)

It is most likely that dilatancy during failure in the sand causes the extra penetration resistance at higher penetration velocities. Figure 4 shows the pore fluid pressure under the pile toe (on the left axis), and force in the toe (on the right axis) as a function of pile displacement. The Figure presents the measurement at a moderate velocity (36 mm/s). As expected, the pore fluid pressure initially rises as the pile is displaced. After a relatively small displacement of the pile tip, the pore fluid pressure starts to decrease. This can be explained by dilatancy during failure. When the pile starts moving upward, the pore water pressure decreases even more. This observation suggests an elastic rebound.

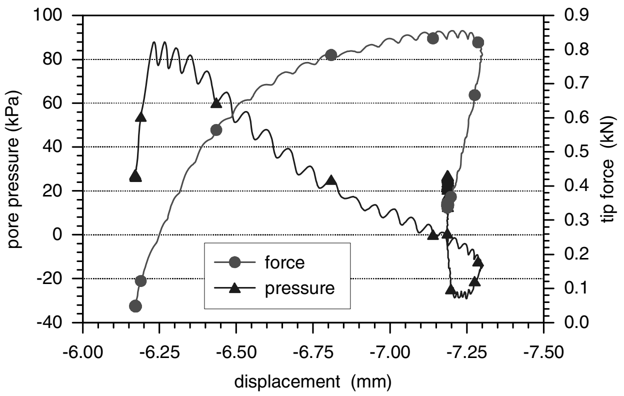

Figure 4. Relationship between pore pressure at the pile tip and penetration force as a function of pile displacement, Test 2 penetration velocity is 36 mm/s.

Figure 5 shows similar graphs as Figure 4, but where the test is carried out with a high velocity (300 mm/s). The pore water pressure shows similar behaviour as in the test with 36 mm/s (see Figure 4), but the maximum pore pressure is higher and the decrease due to dilatancy is less. This can be explained by the fact that the dilatancy occurs in a shear zone, which is close to the pile tip. The transducer in the tip is at some distance from this zone. We therefore only observe the combined effects of pore water generation, pressure wave propagation, and dissipation due to pore fluid flow in the soil (van Tol et al, 2008).

The generation of pore pressure may lead to an additional force against the pile toe, and may influence soil strength due to the change of normal effective stress. The first effect is small. The maximum observed excess pore pressure of 200 kPa corresponds with an extra penetration resistance of 0.02 kN. This is an increase of 5 %, while 50 % is observed (see Figure 2 and Figure 3). When the maximum soil resistance is reached, the measured pore water pressure is even smaller. From these observations, we conclude that the influence of the pore pressure on the strength of the sand dominates the increase of the bearing capacity with increasing loading velocity.

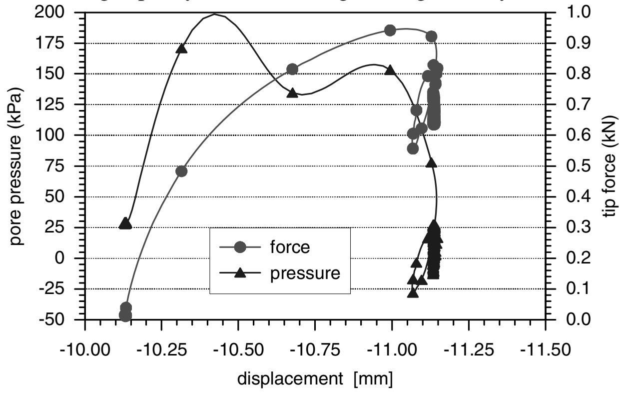

Figure 5. Relationship between pore pressure at the pile tip and penetration force as a function of pile displacement, Test 2 penetration velocity is 300 mm/s.

3 T-BAR TESTS

3.1 *Prototype situation*

Sheet pile constructions can be stabilized with anchors, see Figure 6. This structure is used in The Netherlands in dike-strengthening projects. The sheet pile makes it possible to protect existing buildings behind the dike while increasing the height of the dike. If this type of construction is used on soft soil, the extra weight of the strengthened dike will lead to extra settlement in the soft soil layers underneath. This in turn leads to an extra loading condition for the anchors. The anchor rod bends due to the settling soil, leading to extra tension in the rod. To calculate the extra loading, it is necessary to know the force perpendicular to the rod that is exerted by the clay on the anchor rod during settlement. This is a well-known problem for an undrained situation with inorganic clay such as kaolin (House et.al. 2001). However, for slow deformations and organic clay it was anticipated that the force would be less due to creep of the clay.

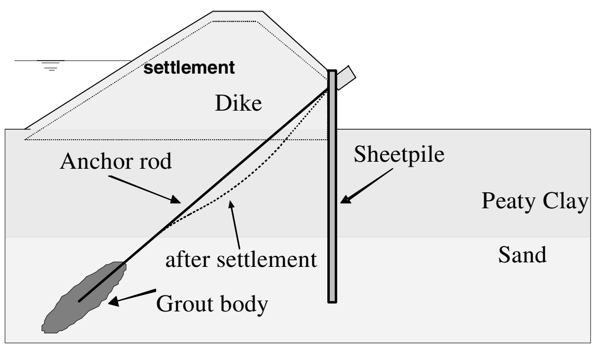

Figure 6. Sheet pile construction stabilized with grout anchor for dike-strengthening projects

3.2 *Tests performed*

Set-up

A series of tests were performed in a clay layer of 0.1 m which was consolidated using a consolidation stress of 100 kPa. At the end of the consolidation, T-bar elements that simulate part of the rod were penetrated into the clay while the consolidation stress was kept constant. The set-up is shown in Figure 7. The penetration velocity ranges from 10 mm/month to 1 mm/sec.

Tests performed

Tests were performed with Speswhite kaolin clay and with reconstituted organic clay from a location where the structure shown in Figure 6 will be constructed. The properties of the clay types used are summarized in Table 2

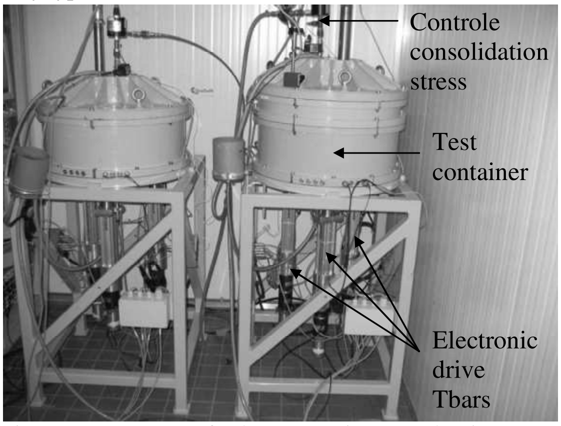

Figure 7. Test set-up of T-bar penetration tests showing test container filled with clay and electronic drive of the three T-bars

Table 2. Properties of Speswhite koalin clay and 'Oostvaarders plassen' clay

Clay properties	Speswhite Koalin clay	"Oostvaarders plassen" clay
Unit weight constituent	26.1 kN/m^3	25.3 kN/m^3
Liquid limit	69%	113%
Plasticity index	31%	84%
Plastic limit	38%	29%
γ at W = 50%	17 kN/m^3	14 kN/m^3
S_u/σ'_v[1]	0.21	0.37
Friction angle (*φ*)	23.0°	28.4°
Cohesion *(C)*	0 kPa	3.9 kPa
Consolidation coeff.*(C_v)*	1.2.10^{-7} m^2/s	5.0.10^{-9} m^2/s

[1] S_u/σ'_v the undrained shear strength/vertical effective stress

Results

The tests on kaolin clay show that the penetration force increases when the penetration rate is decreased. This phenomenon is also described by House et.al. (2001) and Chung et al (2006). They performed centrifuge tests where the penetration rate of a T-bar was decreased leading to higher penetration resistance, and showed this was due to drained behaviour at low penetration velocities. The penetration force was 3.5 times higher at drained conditions compared with undrained conditions. In normally consolidated clay, an undrained penetration condition means that the effective stress in the clay remains constant and the effective stress in the clay in front of the T-bar or CPT will increase during a drained penetration. We found the same phenomenon in our tests, but the factor was only 2.

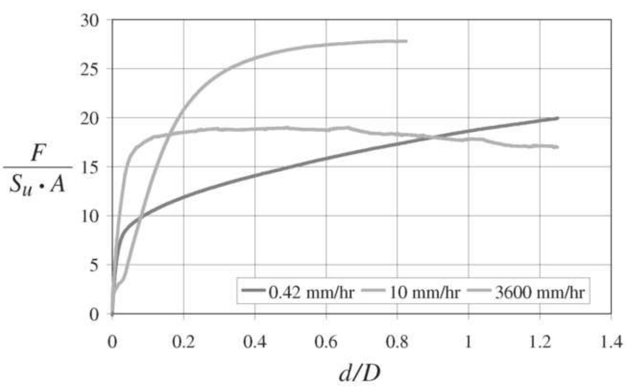

Figure 8. Diagram for different penetration rates of T-bar penetration in 'Oostvaardersplassen' clay, with force *F* normalized with the S_u and the area of the T-bar against the T-bar penetration *d* normalized with the diameter *D* of the T-bar.

The organic clay showed a different behaviour, see Figure 8. At a low penetration rate such as 0.42 mm/hour, the clay shows drained behaviour and does not reach a stable failure mode as seen in the tests on kaolin clay. At these low penetration rates, the force continues increasing with the penetration depth. In the test with a penetration rate of 10 mm/hour, the clay reaches a stable failure mode after *d/D* is equal to 0.7. The clay behaves partially drained at this rate. When the penetration rate is increased to 3600 mm/hour, the clay will behave undrained and a stable failure mode is reached when *d/D* is equal to 0.2.

In contrast with tests on kaolin clay, the penetration resistance does not decrease in all situations with an increasing penetration rate. In the penetration range of $d/D < 0.2$, the highest penetration rate corresponds with the highest penetration force, whilst in the range $d/D < 0.9$ the lowest penetration rate corresponds with the lowest penetration force. It is likely that elastic behaviour plays an important role. Young's modulus seems to be lower in drained conditions. A relatively larger penetration is therefore necessary to reach the plastic failure condition with a constant penetration resistance for ongoing T-bar penetration.

4 DISCUSSION AND CONCLUSION

In both sand and clay, the penetration resistance depends on the penetration rate. There are virtually no viscous effects for sand, and the rate dependency is almost purely dominated by the effects of pore pressure. Undrained penetration of a pile in sand (as may occur during a rapid load test) leads to dilatant behaviour in the sand. This dilatancy leads to decreased pore pressures and an increase in effective stress compared to drained behaviour. The penetration force is consequently higher in an undrained test.

Penetration in kaolin clay shows the opposite mechanism. As long as the penetration rate is fast enough, the behaviour will be undrained and the effective stress in the clay will not change because there is no dilatancy in the normally consolidated kaolin clay. At very low penetration rates smaller than 0.45 mm/s, there will be drained behaviour in kaolin clay. This leads to higher effective stresses in front of the T-bar, and therefore higher penetration forces.

The penetration behaviour of organic clay is different from the behaviour of inorganic clay. This is probably due to the lower Young's modulus of organic clay, which prevents the clay reaching a 'stable failure' mode within the limits of the experimental set-up. The viscous behaviour of the clay also seems to play an important role.

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