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Loading Rate Effect on Piles in Clay from Laboratory Model Tests

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Abstract. The strength of cohesive soils is affected by the rate at which the external load is applied. Bearing capacity of foundations is also affected by the rate of loading since it is a function of the shear strength of soils. In this study, the influence of loading rate on the axial capacity of piles in clay is experimentally investigated. Two series of model tests were carried out using a steel model pile having a diameter of 30 mm under different loading rates. The first group of tests was performed under axial compression loads, while the model pile was subjected to axial uplift loads in the second set of tests. In addition, consolidated undrained triaxial tests were performed under the same loading rates used in the axial capacity tests. Compressive and uplift axial capacities of the model pile were found to increase as the loading rate increases. The relationship between time to failure, and both the undrained shear strength of the clay and the axial capacity of the model pile can be represented by a straight line on a log-log plot.

Introduction

The strength of cohesive soils is affected by the rate at which the external load is applied. Laboratory studies have shown that the undrained shear strength of clay increases as the rate of loading increases [1-7]. It was found from the conventional triaxial tests of different types of clays that the undrained shear strength increased by about 10% for every 10-fold increase of strain rate [8]. Because the bearing capacity of the foundation is a function of the shear strength of the soil, the capacity will also depend on the rate at which the external load is applied.

A review of the literature indicates that research into the effect of rate of loading on the bearing capacity of both shallow and deep foundations in sand and clay is scanty. The limited studies regarding the effect of loading rate on the axial capacity of piles showed that the loading rate affects the axial capacity of piles in clay [9-11].

Pile load tests constitute the most method of determining the behavior of a pile in the field. However, the high cost of conducting full-scale pile tests in the field and the inherently high variability of the field conditions make them impractical for research purposes. Therefore, model tests are usually used for investigating the behavior of piles. There are numerous pile load test methods that have been reported in the literature [12-17]. These methods can be grouped into two main types: the controlled stress tests and the controlled strain tests. The most common methods are the slow maintained-load (SM) test, the quick maintained-load (QM) test and the constant rate of penetration (CRP) test.

In the SM and QM tests, incremental loads are applied to the pile and the resulting deformations are recorded. During the quick test each increment of load is applied at relatively short time intervals compared to the slow test. In the constant rate of penetration test, the pile is made to penetrate the soil at a constant rate, while the load applied to maintain the specified rate of penetration is continuously measured. The constant rate of penetration test has many advantages such as speed, economy and practicality, especially for friction piles, when it is compared with the other tests [17]. Consequently, it was selected and used in this study.

This paper describes an experimental investigation of the behavior of a model pile in clay subjected to axial compressive and uplift loads under various loading rates. In many studies of piles, the uplift loading is applied after the pile has first been tested to failure in compression. The compression test alters the state of stress and the strength along the pile shaft, which may result in smaller values of the measured uplift capacity of the tested piles [18]. So, two series of model tests were carried out in this study using a model steel pile having a diameter of 30 mm. The first group of tests was performed under axial compression loads using five different loading rates. The model pile was subjected to axial uplift loads to failure under the same loading rates in the second set of tests. In addition, consolidated undrained triaxial tests were performed under different loading rates, same as those used in the model pile tests, and the corresponding values of the shear strength of the soil were found.

Effect of Rate of Loading on Bearing Capacity of Piles in Clay

A test pile is in general loaded to failure within a period of several hours to days. During the structure's life, the maximum load may be however applied to the piles years after installation. As well, the period of a critical wave could be less than 10 seconds during a storm. Therefore, the pile capacity measured during load tests would be different from the actual pile capacity. Because shear strength of clay depends on the loading rate, the pile capacity in clay will most likely depend on the loading rate. So, models proposed for the relationship between the shear strength of soils and the time to failure may be used for comparing the bearing capacity of the foundation at different loading rates [19]. One model proposed for comparing bearing capacity of foundations leads to a straight line on a semi-log plot [20] and can be expressed as:

$$\frac{Qu\,(1)}{Qu\,(2)} = 1 + F \log\left[\frac{t(1)}{t(2)}\right] \tag{1}$$

where $Q_u(1)$ and $Q_u(2)$ are the ultimate pile capacities corresponding to times to failure t (1) and t (2), respectively, and F is a constant depending on the type of soil. Bea [21] found that the values of F are 0.01 to 0.03 for sand, 0.02 to 0.07 for silt and 0.02 to 0.12 for clay.

A second model relating pile capacities at different loading rates leads to a straight line on a log-log plot [10]. This model can be expressed as:

$$\frac{Qu\,(1)}{Qu\,(2)} = \left[\frac{t(1)}{t(2)}\right]^{-n} \qquad (2)$$

where $Q_u(1)$ is the ultimate pile capacity corresponding to a time to failure of t (1) and $Q_u(2)$ is the ultimate pile capacity corresponding to a time to failure of t (2), n is a coefficient to be found from the load tests data. Briaud and Garland [10] found that the value of n ranges from 0.02 to 0.10 with an average value of 0.06.

There are relatively few studies regarding the effect of loading rate on the axial capacity of piles in clay. Kraft et. al [9] found that the ultimate pile capacity of piles embedded in clay soil in the field increased by 40% to 75% when the loading rate was increased by about three orders of magnitude. Tang [22] proposed correction factors for the uncertainties caused by many parameters that affect the prediction of the axial pile capacity in clay. Among those factors, he introduced a correction factor to account for the rate of loading effects. The correction factor is about 1.55 with respect to the pile capacity measured from the conventional tests.

Horvath [11] performed model pile tests in clay using three different loading methods, namely, the constant rate of penetration (CRP) tests, the quick maintained loading (QML) tests, and the quick continuous loading (QCL) tests. The total test durations were 10 to 1000 seconds for the QML tests and 0.1 second for the QCL tests. The CRP test was used as a reference test where the average test duration (time to failure) was about 15 seconds. He concluded that the ultimate pile capacity of the model pile in clay increases with increasing loading rate and obtained a semi-logarithm regression relationship between the pile capacity and the time to failure similar to equation (1).

Experimental Apparatus and Procedure

Experimental setup

Figure 1 shows the schematic diagram of the laboratory model test setup. Both series of model tests (compressive and uplift) were carried out using this setup. The experimental setup consists of a soil container, a model steel pile and loading equipment. The vertical load was applied to the pile using a testing machine, which provides a constant rate of vertical displacement. A proving ring and a dial gauge were used for measuring load and pile displacement, respectively.

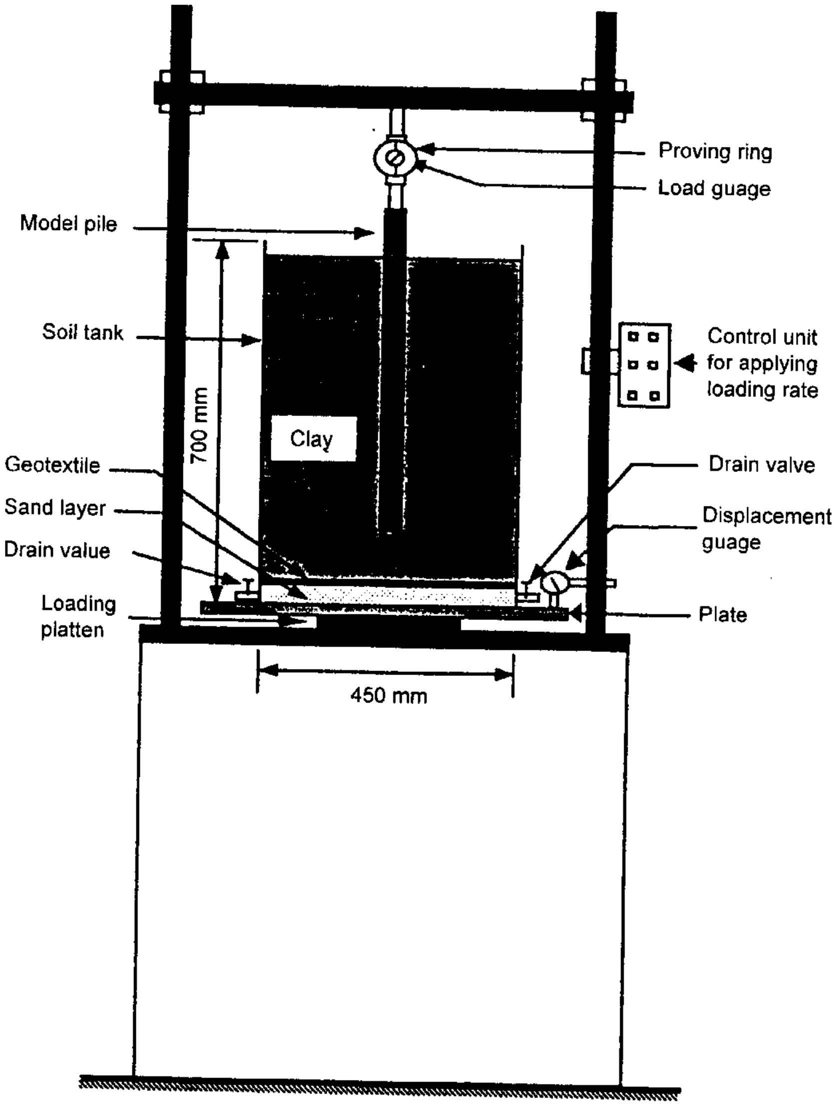

Fig. 1. Schematic diagram of the laboratory model test setup.

Soil container and model pile

A cylindrical steel tank having outside and inside diameters of 450 mm and 438 mm, respectively with a height of 700 mm was used as the soil container. Two holes in opposite sides were made about 10 mm from the bottom of the tank, and two valves were installed in them to control water drainage during both the consolidation stage and the pile test stage. An external square steel plate with a thickness of 6 mm was externally welded to the tank bottom and connected to the loading platen of the testing machine.

A smooth solid steel pile having a diameter of 30 mm was used in this study. This size could be considered as a representative size for model pile testing as suggested by Vesic [23]. It was also selected to minimize the boundary effects of the soil container on the results of the conducted tests. The soil tank has lateral boundaries (the soil tank diameter is about 15 pile diameters) that satisfy the criteria implied by the zone of influence determined from the elastic theory [24], as well as, the influence zone found by other investigators [25-27]. Therefore, it is expected that there is a negligible boundary effect in the experimental setup.

Properties of the investigated soil

The soil used in this study is a homogeneous soil obtained from Al-Watania bricks factory in Riyadh, capital of Saudi Arabia. It is commercially known as a fire clay. Laboratory tests including specific gravity, liquid limit, plastic limit, and grain size analysis were performed in accordance with ASTM standard procedures to find the soil properties. The grain size distribution of the clay is shown in Fig. 2. The clay had a unified soil classification of CL (silty clay). The average properties of the clay are shown in Table 1.

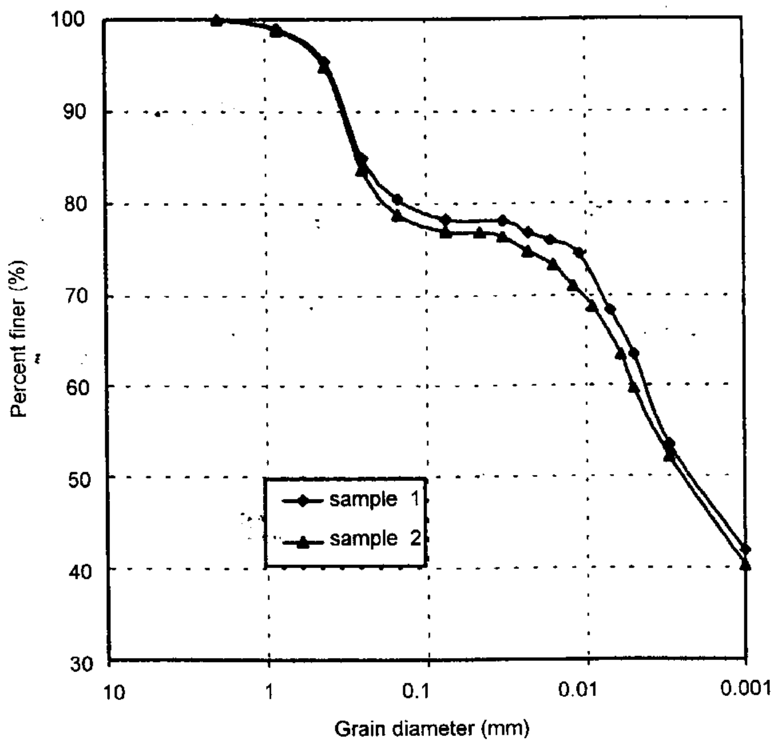

Fig. 2. Grain size distribution curve for the clay.

Table 1. Average geotechnical properties of the clay

Parameter	Value
Color	Brown
Specific gravity of solids	2.80
Sand content (%)	22
Silt content (%)	30
Clay content (%)	48
Consistency limits	
Liquid limit (%)	36
Plastic limit (%)	22
Plasticity index (%)	14
Classification	
Unified soil system	CL
AASHTO classification system	A-6

Experimental procedure

Each model test consists of two stages; consolidation stage and pile loading stage. The clay was oven-dried and the calculated weight to get an initial dry unit weight of 14.60 kN/m^3 was thoroughly mixed with the required amount of water to obtain an initial moisture content of 30 %. Thereafter, the mixture was placed in a plastic bag to avoid loss in moisture content, and kept in room temperature for about 24 hours to obtain a homogeneous mixture.

The consolidation process was performed in a specially designed loading frame using the lever-type arrangement shown in Fig. 3 with a lever arm ratio of 1:10. A layer of poorly graded sand was placed at the bottom of the soil tank to serve as a drainage layer. The prepared clay was placed by hand in the soil tank in three layers, each layer is 180 mm thick. The first clay layer was placed over a geotextile separating the clay from the sand layer. After that, an upper plate having a diameter of about 435 mm and a thickness of 15 mm was placed on the top of the clay. Several holes with diameters of about 5 mm were punched into the plate, as well as, sheet filter papers were placed along the sides of the tank and between the clay and the loading plate to speed up the consolidation process. The drainage valves were then opened and the first load increment of 10 kPa was applied.

Soil deformation was monitored and readings of settlement were taken at certain time intervals until the relationship between settlement and the logarithm of time became nearly horizontal. The settlement of the clay was measured by means of two dial gauges, which were connected to the upper plate (Fig. 3). Consequently, the second load increment of 20 kPa was applied and after the consolidation of the clay was finished, a third increment of 40 kPa was applied. The last load increment was 80 kPa.

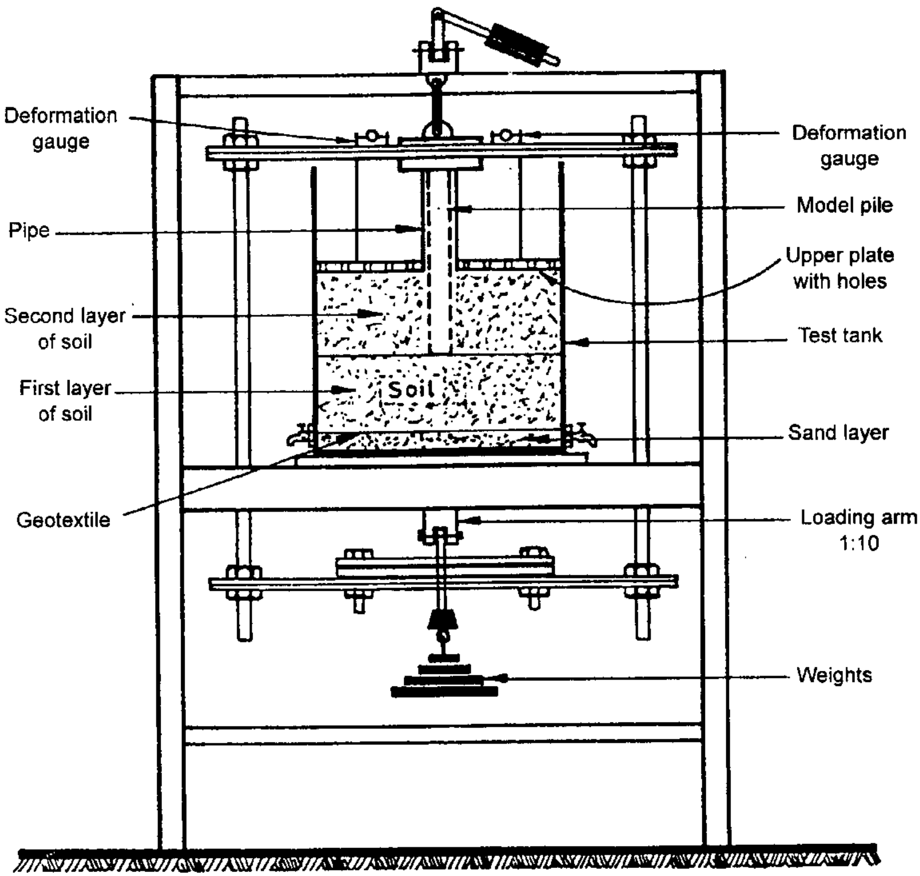

Fig. 3. Consolidation loading frame.

After the consolidation of the first layer of the clay had been completed, the model pile was held vertically at the center of the tank above the first layer. Then the second layer of the clay was placed around the pile, and the upper plate was put on this layer. The plate has a hole in the center with a diameter larger than the pile diameter in order to prevent the pile from interfacing with the plate during the consolidation process. A steel pipe was inserted to connect the plate with the loading arm to transfer the consolidation load to the soil without affecting the pile (Fig. 3). As well, the second layer was consolidated in the same manner. Finally the last layer was placed on the second layer and subjected to the same consolidation loads. The consolidation process of the three layers took about thirty-six days for each test. Although, the pile is initially sitting on the smooth surface between the first and the second layers of the clay, it is expected that the pile will slightly penetrate into the first layer due to the consolidation of the subsequent layers of the clay.

Pile compression tests

After the consolidation of the clay had been completed, the consolidation loads were removed, and the soil tank was carefully removed from the consolidation frame and mounted on the testing machine. Thereafter, the two valves at the bottom of the soil tank were closed and the specified loading rate was set. The model pile was subjected to an axial compressive load until failure occurs. A total of five compressive model tests were performed under five different loading rates, namely, 1 mm/min., 0.5 mm/min., 0.1 mm/min., 0.05 mm/min. and 0.01 mm/min. The load applied to the pile was measured by a proving ring, and the pile head displacement was measured by a dial gauge.

Uplift model tests

Similar to the compressive model tests, the uplift load was applied to the pile after the consolidation of the clay had been completed, and the soil tank was carefully removed from the consolidation frame and mounted on the testing machine. In this set of tests, the axial uplift load was applied to the model pile using the same loading rates used in the compression load tests (i. e. 1 mm/min., 0.5 mm/min., 0.1 mm/min., 0.05 mm/min. and 0.01 mm/min).

Triaxial tests

A total of five consolidated undrained triaxial tests were conducted under different loading rates same as those used in the pile load tests. All samples had the same initial dry unit weight of 14.6 kN/m^3 and initial moisture content of 30%, the same initial conditions of the clay in the model pile tests. Soil samples were statically compacted to the desired dry unit weight, placed in the triaxial apparatus and left for about 24 hours to reach saturation. The saturation was checked by applying back pressure and measuring the pore water pressure. Thereafter, samples were consolidated in the triaxial cell for about 24 hours under a confining pressure of 80 kPa which is equal to the final consolidation pressure applied to the soil in the soil tank. The drainage valve was then closed and the shearing process was initiated.

Test Results and Discussion

Load-displacement response

The load-displacement curves are shown in Figs. 4 and 5 for the compression and uplift model tests, respectively. It could be seen from these figures that the load-displacement curves have peak values for both the compression and uplift tests from which the pile load reduces with further displacement. The pile head displacement needed to mobilize the ultimate compressive load ranges from 2 mm to 3 mm (about 10% of pile diameter) which is in agreement with the values suggested by Vesic [23]. On the other hand, the ultimate uplift loads were found to be at pile head displacement of 1 mm to 2 mm, which is slightly lower than the displacement needed for the compressive loads. The lower values of pile head displacement at failure for the uplift loads might be attributed to the fact that only the side resistance is mobilized in the uplift tests, while both the side and end bearing resistances are mobilized in the compression tests.

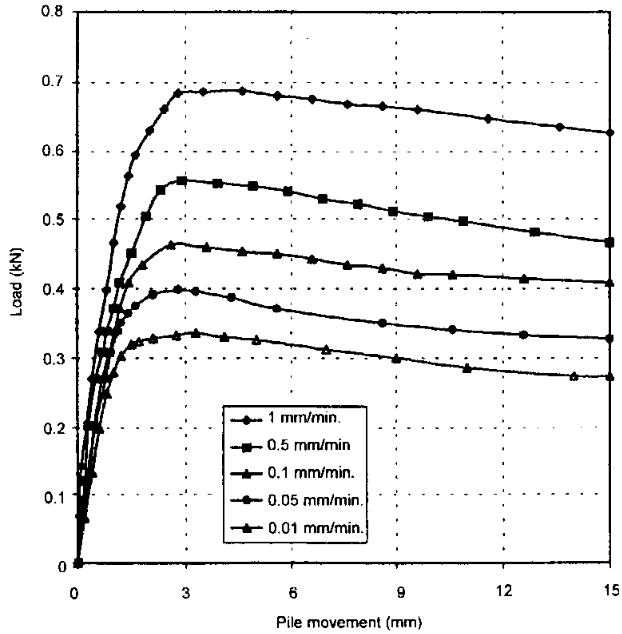

Fig. 4. Load-displacement curves for compression tests.

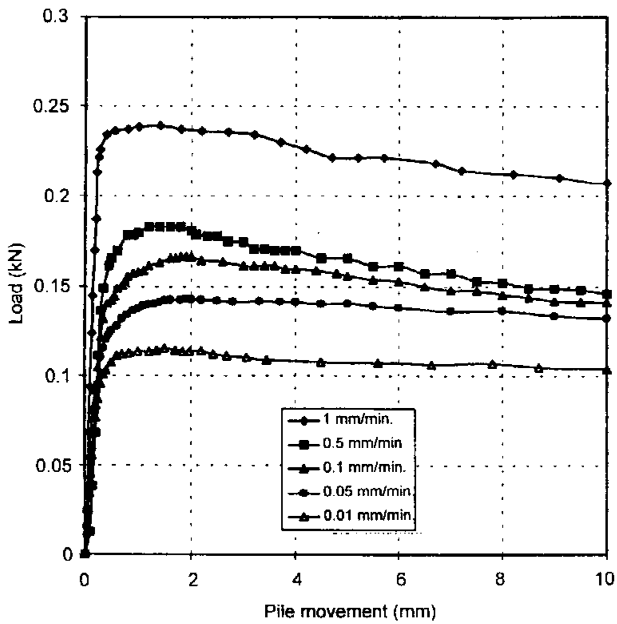

Fig. 5. Load-displacement curves for uplift tests.

The pile head displacement at failure is essentially the same for the slow and the fast tests, while only the ultimate capacity (both compressive and uplift) changed. It means that the loading rate has a negligible influence on the magnitude of the pile head displacement at failure. This is in agreement with the findings of Audibert and Dover [28].

Effect of loading rate on pile-soil stiffness

The load-displacement response was further evaluated in terms of the pile-soil stiffness. The slopes of the initial straight line of the load-displacement curves, stiffness, for both the compression and uplift tests are found from Figs. 4 and 5, and plotted against the time to reach the end of the straight line in Fig. 6. The relationship is obtained by fitting of the experimental data resulting in the following equation:

$$k = 0.44\,(t)^{-0.08} \tag{3}$$

where k is the pile-soil stiffness, and t is the time to reach the end of the straight line of the load-displacement curve. The pile-soil stiffness of both compression and uplift tests was found to increase as the rate of loading increases.

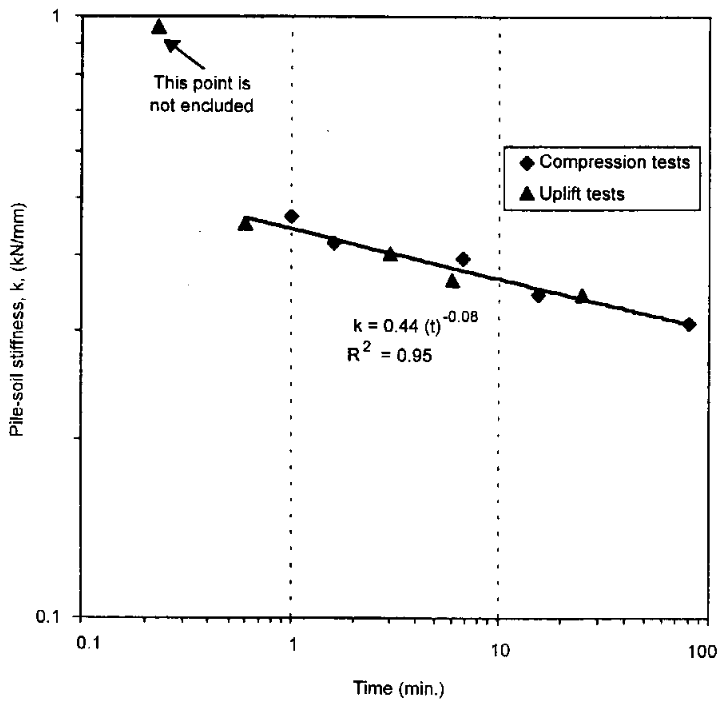

Fig. 6. Effect of loading rate on pile-soil stiffness.

Effect of loading rate on ultimate pile capacity

The ultimate loads (peak values) obtained from the compression (Fig.4) and uplift (Fig.5) tests were plotted against time to failure in Fig. 7. The relationship between the axial pile capacity and time to failure can be approximated by a straight line on the log-log plot. The gain in pile capacity is given by the following equations:

$$Q_u = 0.76\ (t)^{-0.15} \quad \text{(compression tests)} \tag{4}$$

$$Q_u = 0.22\ (t)^{-0.12} \quad \text{(uplift tests)} \tag{5}$$

Equations 4 and 5 are similar to equation 2, where the value of the exponent n is about 0.15 for the compression tests and about 0.12 for the uplift tests. This finding is in agreement with the data reported by Briaud and Garland [10], however, the values of the exponent n are slightly higher than 0.10 obtained by them.

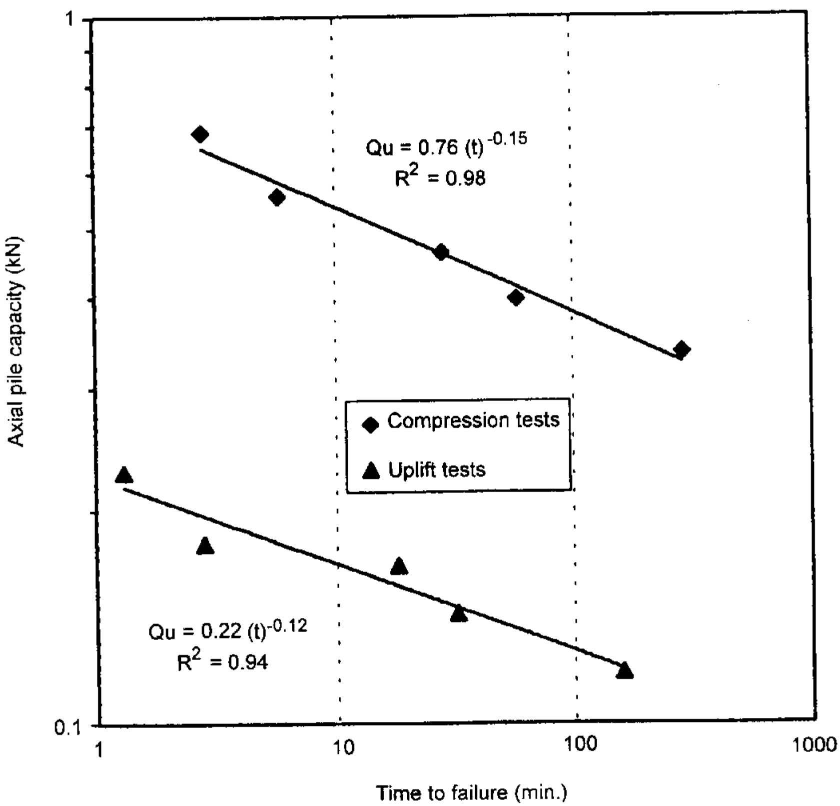

Fig. 7. Effect of loading rate on axial pile capacity.

It could be seen from Fig. 7 that the effect of loading rate on both the compressive and uplift pile capacities is significant. The effect of loading rate on the compressive and the uplift pile capacities is approximately the same because the value of the exponent n for the compressive capacity (0.15) is slightly higher than that for the uplift capacity (0.12). A possible explanation for this behavior is that piles in clay derive their capacity primarily from side resistance [19]. Both the compressive and uplift capacities of the model pile increase as the loading rate increases. For example, the compressive pile capacity is about 0.33 kN for the loading rate of 0.01 mm/min. It is almost doubled about 0.68 kN, when the loading rate increased to 1 mm/min., i.e. approximately a twofold increase in compressive pile capacity for an increase in the load rate by two orders of magnitude. The uplift capacity under a loading rate of 0.01 mm/min. is about 0.12 kN. It increases to about 0.23 kN, when the loading rate is 1 mm/min.

The test results showed that the increase in pile-soil stiffness with increasing loading rate is less than the increase in the axial pile capacity, which is in agreement with the findings of Kraft *et. al.* [9]. The values of the exponent n are 0.08, 0.12, and 0.15, for the pile-soil stiffness, uplift capacity and compressive capacity, respectively.

Effect of loading rate on undrained shear strength of the clay

The values of the undrained shear strength obtained from the triaxial tests under the five loading rates versus time to failure are presented in Fig. 8. The relationship could be represented by a straight line on the log-log plot. The relationship is given by:

$$S_u = 53.80\,(t)^{-0.10} \tag{6}$$

It is clear that undrained shear strength of the tested soil increases as the loading rate increases. The obtained values of the undrained shear strength were used to back calculate the value of the adhesion factor α.

Adhesion factor α

The values of the adhesion factor α were back calculated from the measured axial pile capacity and the measured undrained shear strength of the clay using the following equations. The ultimate capacity of a vertical straight-shafted pile in cohesive soil is calculated from the well-known equations:

$$Q_u = Q_p + Q_s - W_p \qquad \text{(compressive capacity)} \tag{7}$$

$$Q_u = Q_p + Q_s + W_p \qquad \text{(uplift capacity)} \tag{8}$$

where Q_u is the ultimate capacity of the pile, Q_p is the end-bearing capacity, Q_s is the side capacity and W_p is the weight of the pile.

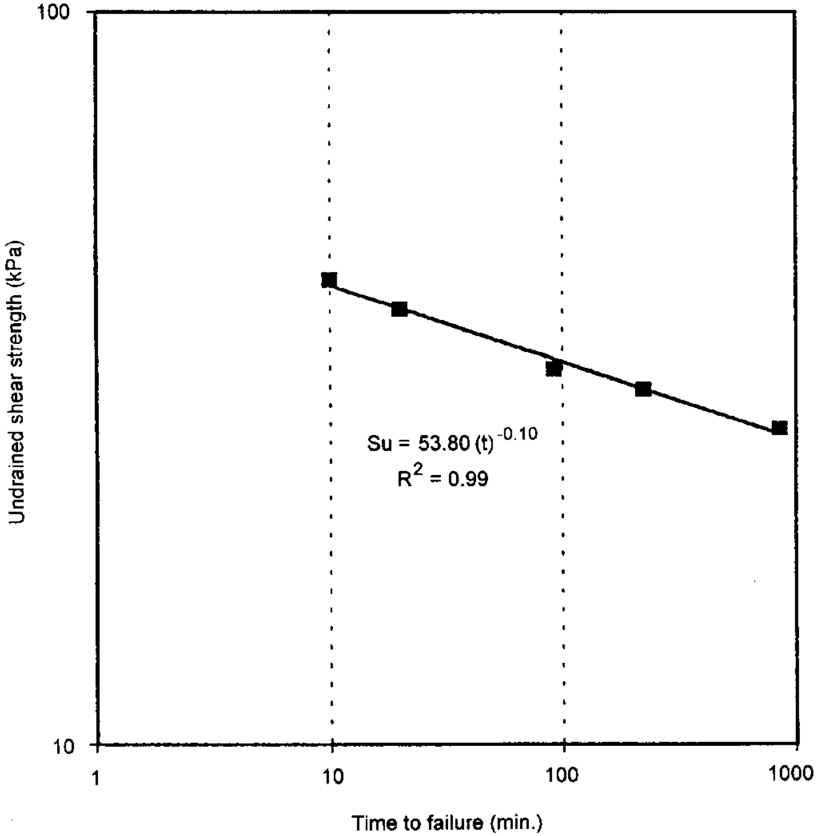

Fig. 8. Effect of loading rate on undrained shear strength of the clay.

The end-bearing capacity is calculated from the following equation:

$$Q_p = A_p * q_p = A_p * S_u N_c \tag{9}$$

where A_p is the cross-sectional area of the base of the pile, S_u is the undrained shear strength of soil and N_c is the bearing capacity factor, which is generally equal to 9. The end-bearing capacity is often assumed to be zero for piles in uplift [18].

The α method is often used to calculate the side capacity of piles in cohesive soils [29]. The following equation is used:

$$Q_s = A_s * f_s = A_s * \alpha S_u \tag{10}$$

where A_s is the side area of the pile, S_u is the undrained shear strength of soil and α is the adhesion factor.

Table 2 shows the back-calculated values of the adhesion factor α from both the compression and uplift test results. The values range from 0.21 to 0.33 with an average value of 0.26 for compression tests. However, they range from 0.15 to 0.18 with an average value of 0.16 for uplift tests. The obtained values of α are lower than those reported in the literature. Das [30] suggested that $\alpha = 0.2$ for steel piles uplifted in clay with undrained shear strength greater than 27 kN/m^2. This recommended value is slightly higher than the average value (0.16) obtained from the uplift tests. The possible reason for obtaining these low values, especially those from the compression tests, might be caused by the smooth surface of the steel pile and the placement procedure of the model pile which give very low horizontal stresses.

Table 2. Back calculated values of the adhesion factor α

Loading rate	Compression tests	Uplift tests
1 mm/min.	0.33	0.18
0.5 mm/min.	0.27	0.15
0.1 mm/min.	0.27	0.17
0.05 mm/min.	0.23	0.16
0.01 mm/min.	0.21	0.15
Average value	0.26	0.16

Summary and Conclusion

The influence of loading rate on the compressive and uplift capacity of a steel model pile embedded into clay was investigated. Two series of model tests were carried out using a model pile having a diameter of 30 mm under different loading rates. The first group of tests was performed under axial compression loads, while the model pile was subjected to axial uplift loads in the second set of tests. Consolidated undrained triaxial tests were performed under the same loading rates used in the pile model tests and the corresponding values of the undrained shear strength were found. The values of the adhesion factor α were back calculated using the values of the undrained shear strength obtained from the triaxial tests and compared with those reported in the literature.

Based on the test results, the following observations and conclusions are presented:

1. Loading rate has a significant influence on both the compressive and the uplift capacity of the model pile. Increasing the loading rate results in an increase of the pile capacity. The relationship between the axial pile capacity and the time to failure can be represented by a straight line on a log-log plot. The values of the

exponent n are about 0.15, and about 0.12 for the compressive and the uplift capacity, respectively.

2. Loading rate has a negligible influence on the magnitude of the pile head displacement at failure for both the compression and the uplift tests.

3. The increase in pile-soil stiffness with increasing loading rate is less than the increase in the axial pile capacity both in compression and uplift tests.

4. Undrained shear strength of the tested soil increases as the loading rate increases. Similar to the effect of loading rate on the axial pile capacity, the relationship between the undrained shear strength and time to failure can be represented by a straight line on a log-log plot. However, the value of the exponent n is about 0.10.

5. The back-calculated values of the adhesion factor α from both the compression and uplift tests are lower than those reported in the literature which might be caused by the smooth surface of the steel pile and the placement procedure of the model pile which give very low horizontal stresses.

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تأثير معدل التحميل على الخوازيق في الطين من تجارب نماذجية معملية

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ملخص البحث. تتأثر قوة القص للتربة المتماسكة بالمعدل الذي يطبق فيه الحمل الخارجي. كما أن قوة تحمل الأساسات تتأثر أيضا بمعدل التحميل لأنها تعتمد على قوة القص للتربة. وفي هذه الدراسة ، تم بالتجربة بحث تأثير معدل التحميل على قوة التحمل الرأسية للخوازيق في الطين. وتم إجراء سلسلتين من التجارب المعملية باستخدام نموذج خازوق حديدي قطره ٣٠ ملم وتحت معدلات مختلفة من التحميل. وتم عمل التجارب في المجموعة الأولى تحت أحمال الضغط الرأسية بينما تم تحميل الخازوق تحت أحمال الشد الرأسية في المجموعة الثانية من التجارب. بالإضافة إلى ذلك ، تم إجراء تجارب الاختبار الاندماجي المحبوس في جهاز الضغط ذو الثلاثة محاور تحت معدلات التحميل التي تم استخدامها في تجارب قوة تحمل الخازوق الرأسية نفسها. ولقد ، وجد أن قوة تحمل الخازوق الرأسية للضغط والشد تزيد بازدياد معدل التحميل. كما أن العلاقة بين زمن الانهيار وكلٍ من قوة القص للتربة وقوة تحمل الخازوق الرأسية يمكن تمثيلها بخط مستقيم على ورقة رسم بياني لوغاريثم - لوغاريثم.