STATENS GEOTEKNISKA INSTITUT

SWEDISH GEOTECHNICAL INSTITUTE

No. 42

SÄRTRYCK OCH PRELIMINÄRA RAPPORTER

REPRINTS AND PRELIMINARY REPORTS

Supplement to the "Proceedings" and "Meddelanden" of the Institute

Contributions to the Seventh International Conference on Soil Mechanics and Foundation Engineering, Mexico 1969

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STOCKHOLM 1971

PREFACE

The Swedish contributions to the Seventh International Conference on Soil Mechanics and Foundation Engineering in Mexico 1969 are collected in this publication. To the first paper are added the Swedish contributions to the discussions at the Specialty Session on negative skin friction on piles in clay.

The Institute wants to thank Mr L.Bernell and Mr K.A.Scherman, Swedish State Power Board, for the possibility to include their paper in the publication.

Stockholm, May 1971

SWEDISH GEOTECHNICAL INSTITUTE

NEGATIVE SKIN FRICTION FOR LONG PILES DRIVEN IN CLAY

LE FROTTEMENT NEGATIF POUR DE LONGS PIEUX ENFONCES DANS L'ARGILE

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SYNOPSIS The paper describes the load distribution in two instrumented pre-cast concrete piles which have been driven through 40 m of clay, 13 m of silt and 15 m of sand. The load is caused by negative skin friction due to reconsolidation of the remoulded clay around the piles after the driving. The soil consists at the test site of homogeneous normally consolidated clay with a water content of around 80 % and an undrained shear strength at the surface of about 2 tons/m^2 and 7 tons/m^2 at the depth of 40 m. Piezometers and settlement gauges were installed in the soil prior to the driving.
A new accurate pile-force gauge has been developed for this project which makes it possible to measure the load and the moment distribution in a pile after driving. Measurements showed that the load in the pile immediately after driving was roughly equal to the weight of the pile itself. During the five months period following the driving the load in the pile increased by about 30 tons at the bottom of the clay layer. This load increase corresponds to a negative skin friction which increases linearly from 0 at the ground surface to about 1.4 tons/m^2 at the bottom of the clay layer. The negative skin friction is equal to 17 % of the undrained shear strength or to 5 % of the average effective overburden pressure.

INTRODUCTION

Thick deposits of soft normally consolidated clays cover large areas in the middle and south western parts of Sweden. Building in these areas are generally supported by end bearing piles which are driven through a soft clay to moraine or rock. However, the general lowering of the ground water table which occurs in the central parts of the large cities causes settlements and an increase of the load in the end bearing piles due to negative skin friction. Great uncertainty exists about the magnitude of the negative skin friction with respect to the undrained shear strength of the soil or the effective overburden pressure, the relationship between settlements and negative skin friction and the effect of pile driving on the negative skin friction. An investigation was therefore initiated in 1966 in order to answer some of these questions.

A robust and accurate pile force gauge has been developed for this project. The pile force gauge was designed to resist the stress conditions during the driving of a pile. The conventional system to measure the negative skin with a series of steel rods has the disadvantage that only the changes which take place after the driving can be measured. Thus the stress conditions in the pile immediately after the driving are unknown. An additional uncertainty in the calculations is that the value of the modulus of elasticity for concrete is uncertain and may vary appreciably. These variations affect the accuracy of the results. Two instrumented reinforced concrete piles were driven in June 1968. The pore water pressures, the soil movements and the negative skin friction which develop during the driving and during a five month period after the driving are described in this paper.

TEST PROGRAM

The test program consists of three phases. The first phase concerns the distribution of negative skin friction and bending moments during and after the driving of two long unloaded piles which have been driven through a normally consolidated clay into silt and sand. In the second phase an axial load of 80 tons will be placed on the two piles and its effects on the negative skin friction of this load will be studied. In the third phase a 2 meter high gravel fill will be placed over an area 40 by 40 meters around the two test piles. This will cause settlements in the clay and thus negative skin friction along the two piles. It is anticipated that phase three of this investigation will be completed in 1973.

SITE CONDITIONS

The test site is located at the River Göta Älv approximately 20 km northeast of Gothenburg in the southwestern part of Sweden. The soil consists of 40 m of normally consolidated clay which is underlain by silt and sand. At 35 m depth the clay contains silt layers. The undrained shear strength, water content, liquid and plastic limits, fineness number and unit weight are shown in Fig. 1. The natural water content of the clay exceeded the liquid limit down to a depth of about 23 m. The undrained shear strength increased from about 1.5 tons/m^2 at the ground surface to about 5.0 tons/m^2 35 m below the ground surface. The ground water table is located at the ground surface. The percentage of clay size particles smaller than 0.002 mm in the clay is about 80 % down to a depth of 20 m. Between 20 and 30 m the percentage of clay size particles decreases to about 55 %. The sensitivity of the clay varies between 15 and 20.

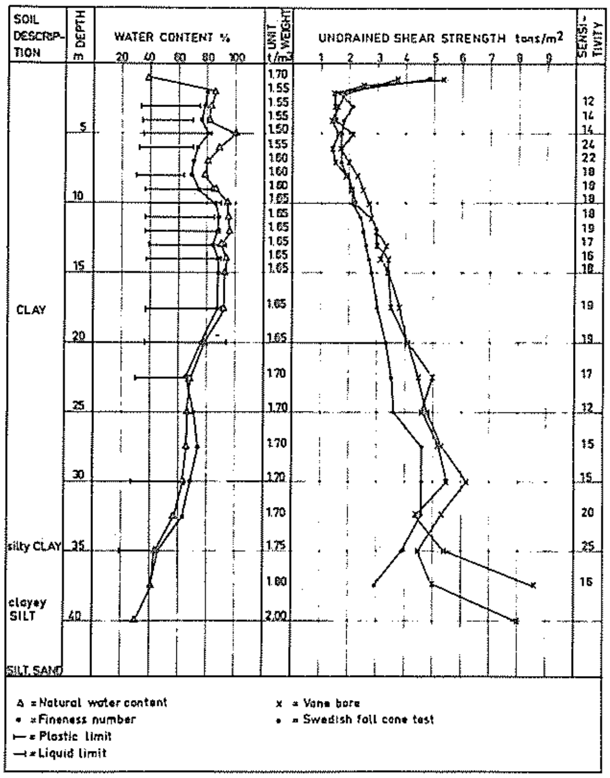

Fig. 1 Soil description

PILE TYPE AND DRIVING DATA

Two precast hexagonal Herkules piles of reinforced concrete with a cross sectional area of 800 cm^2 and a circumference of 106 cm were used for the experiments. Each pile was composed of 11.2 m long segments. The bottom segment was provided with a rock point of hardened steel. The piles were also provided with a center pipe, a thin wall steel pipe with 42 mm diameter inside diameter, in which deformation gauges were inserted after the driving. Also special cable pipes (Ø 8 mm) were placed in the piles for the electrical cables leading from the pile force gauges to the head of the pile.

The nominal concrete cube strength was 500 kg/cm^2. The average measured cube strength was 607 kg/cm^2 28 days after the casting. The reinforcement consisted of six bars with 16 mm diameter and a yield strength of 60 kg/mm^2. The failure bending moment of the pile section exceeded 8.5 tonm. The segments were fastened together in the field by rigid steel joints as the driving proceeded. The strength of the joints exceeded that of the pile segment.

Test pile PI was composed of five pile segments and three pile force gauges and test pile PII of six segments and four gauges. The length of the upper segment of pile PII was 2.0 m. In this pile the lowest gauge was placed right at the pile tip as illustrated in Fig. 2. The location of the pile force gauges are shown in Fig. 3.

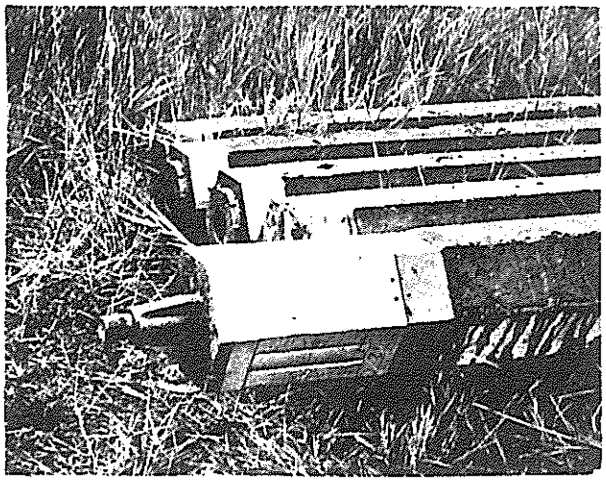

Fig. 2 Photo of pile-force gauge placed at the pile tip. The pile tip is provided with a rock point of hardened steel

The piles were driven with a 4.2 tons drop hammer. The height of fall was 0.3 m when the piles were driven down to a depth of 40 m. Below this depth the height of fall was increased to 0.5 m. The total number of blows required for the driving was about 5000 for pile PI and 4000 for pile PII. The driving of the first pile (PI) was terminated at a depth of 53.1 m when the penetration resistance of the pile was 8 cm per 50 blows. The second pile (PII) had to be driven to a depth of 55.1 m to reach the same final penetration resistance as pile PI (The penetration resistance at 53 m depth was low and therefore an additional 2 m long pile segment had to be added). The driving data indicated that the piles acted as combined friction and end bearing piles.

Pile PI was relatively straight after the driving. Inclinometer measurements (Kallstenius and Bergau, 1961) indicated that the pile tip deviated laterally 1.4 m from its intended position. The minimum radius of curvature of the displaced pile was 340 m. Pile PII was not as straight as pile PI. The pile tip after driving had been displaced 6.2 m away from its intended location. The minimum radius of the pile axis close to the pile point was 170 m. Laboratory tests have indicated that failure by bending will occur at a radius of 50 to 100 m.

INSTRUMENTATION OF PILES

The pile-force gauge used in this project was developed by the A. Johnson Institute for Industrial Research. The gauge is composed of three load cells which are placed between two rigid steel plates.

The load in each cell is measured separately by a system of vibrating wire gauges. This makes it possible to determine the total axial force and bending moment in the test pile at the level of the force gauge. The 0.4 m long force gauges were connected to a pile in the same manner as the pile segments. The pile force gauge was designed to resist the stress conditions which develop during the driving. Laboratory and field tests have indicated that the maximum error in the recorded forces is less than 2 % of the design load. The gauges were designed to resist a tension load of 50 tons and a compression load of 150 tons. These values can, however, be exceeded three times without impairing the function of the gauges. (Fellenius and Haagen, 1969.)

INSTRUMENTATION OF SOIL

Piezometers and settlement gauges were installed two months prior to the driving of the piles. All piezometers but one were type SGI which are provided with a closed oil system. Pore water pressures are read directly on a manometer (Kallstenius and Wallgren, 1956). To measure the pore pressure in the permeable bottom layers an open pipe with a filter tip was used. Each settlement gauge consisted of a number of 2 m sections of flexible steel spring reinforced rubber hoses with 32 mm inside diameter. The steel spring reinforcement allowed the hose to change its length axially but prevented the hose from collapsing when subjected to lateral earth pressure. The hose sections were connected by brass rings. The settlement gauges were placed vertically in pre-drilled holes in the soil. The flexible settlement gauges and the brass rings followed the movements of the soil. It was possible to determine the settlements of the soil from the location of the brass rings with respect to a reference point at the ground surface by lowering a plumb bob inside the hose. When the plumb bob came in contact with the brass rings an electrical circuit was closed which could be observed at the ground surface. With this method it was possible to determine the settlements every 2.0 m with an accuracy of ± 2 mm (Wager, 1969).

Two settlement gauges were installed next to each pile. An additional gauge was installed at some distance away from the pile. The gauges were brought down to a depth of 36 m. The location of the gauges are shown in Fig. 3. The various gauges could not be installed absolutely vertically. However, the deviations were small down to a depth of 10 m.

Three piezometers were installed next to the predetermined pile locations at the depths 9.0, 22.3 and 30.5 m below the ground surface. One additional piezometer was installed at a depth of 28.6 m some distance away from the two pile. At about the same distance from the piles an open pipe with a filter tip was installed at a depth of 45.0 m.

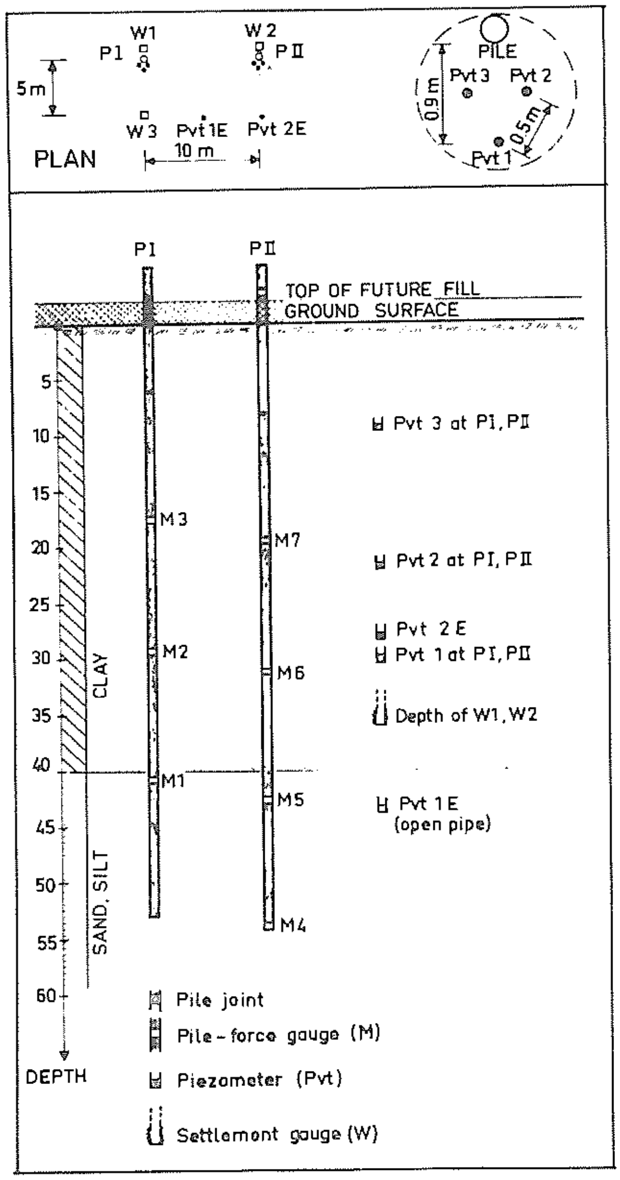

Fig. 3 Location of piles and instrumentation

BEHAVIOUR DURING AND IMMEDIATELY AFTER DRIVING

The driving of the two test piles caused movements in the soil. These were measured close to the two piles and at some distance away from the piles. Also high excess pore pressures were measured.

Settlements. The driving caused the ground surface to heave 20 mm close to the pile as shown in Fig. 4. The heave decreased, however, with increasing depth. Settlements were measured below a depth of 5 to 6 m. The maximum settlement (50 mm) was measured close to pile PII at a depth of 11 m. The observed heave was caused by upward displacements of the soil above the pile tip and the observed settlements probably by downwards displacements at and below the pile tip.

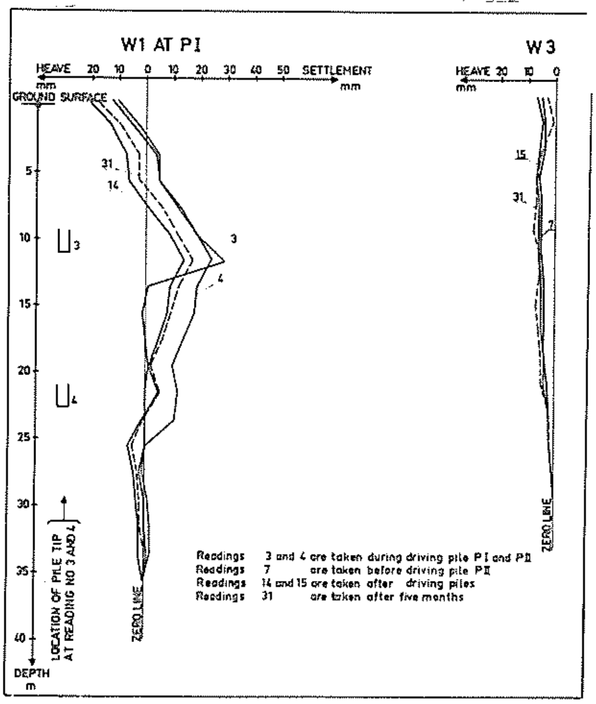

Fig. 4 Vertical movements in the clay during and after driving

The continued driving caused the soil to heave about 10 to 15 mm. At gauge WIII located 5 and 11 m away from the test piles the driving caused a heave of 5 mm at the ground surface. The heave decreased, however, with depth. The displacements of the soil shown in Fig. 4 have been evaluated from the assumption that the soil at the lowest measuring point did not move. The reported values thus represent relative movements within the clay layer. A precision levelling before and after the driving indicated, however, that the lowest measured point of gauge WI had settled 9 mm. The corresponding settlements of gauges WII and WIII were 7 mm and 5 mm, respectively. These settlements are primarily caused by compaction of the silt and sand layers below the clay. The two test piles were driven 13 and 15 m into the bottom layers.

Pore Water Pressure. The pore water pressures measured by all piezometers corresponded prior to the driving of the two test piles to a ground water table at the ground. The driving caused, however, a large increase of the pore water pressure at the gauges located at a depth of 20.3 m below the ground surface. The pore pressure increase at the gauges located at 30.5 m depth was small since these gauges were located several meters away from the two piles. Also the pore pressure increase observed at a depth of 9.0 m was small. All piezometers have afterwards been checked and are functioning properly. The excess pore pressure caused by the pile driving and the dissipation of the excess pore pressures with time are shown in Fig. 5 for the two gauges located close to piles PI and PII at a depth of 20.3 m. The maximum total pore pressure was 40 tons/m^2 at this level. The corresponding total vertical overburden pressure is 32.9 tons/m^2 at the same level. Thus the measured pore pressure exceeded locally the total overburden pressure by 20 %.

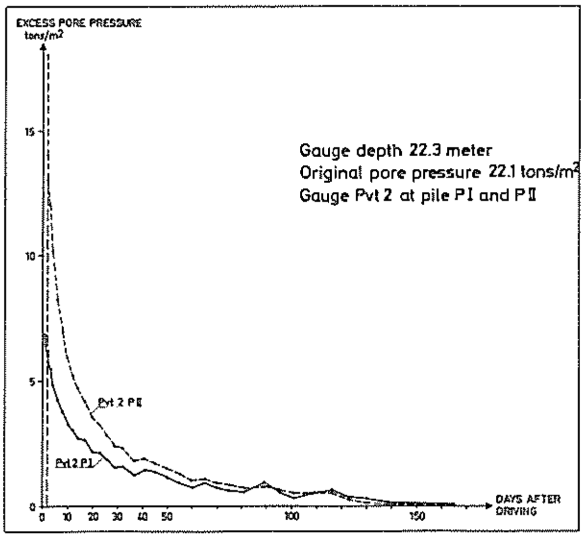

Fig. 5 Excess pore pressure caused by pile driving and its dissipation with time

Forces and bending moments. The force gauges in the piles were read each time a new pile segment was added.

Measurements indicate that the force in the two piles immediately after the driving was roughly equal to or slightly less than the weight of the pile above the gauge. Thus the driving did not cause any axial forces to be "locked" into the piles.

The bending moments in the straight pile PI were small. These varied between 0.4 and 1.3 tonm. Larger values were measured in the bent pile PII. Gauge M5 located 12 m from the pile tip at the boundary between the clay and the underlaying silt and sand indicated a bending moment of 3.2 tonm. This bending moment corresponds to about 35 % of the failure value. The corresponding radius of curvature was 170 m over the length of the gauge.

Gauges M6 and M7 indicated a bending moment of 0.9 and 2.4 tonm, respectively. The corresponding radii were 220 and 190 meters.

BEHAVIOUR AFTER DRIVING

The driving disturbed the clay around the piles. It was anticipated that reconsolidation of the clay would cause settlements of the soil and drag forces in the piles. To study this phenomenon the various instruments were read regularly during the five months period which followed the driving.

Settlements. The settlement gauges indicated that the movements of the soil were small. The recorded settlements varied between one and three millimeters.

Pore water pressure. High excess pore water pressures developed around the driven pile in the clay as indicated by the gauges located at a depth of 20.3 m. The excess pore water pressures dissipated with time. No excess pore water pressures remained 150 days after the driving as can be seen in Fig. 5.

Forces and bending moments. The axial force distribution with time is shown in Fig. 6. At first the axial load in the two test piles increased rapidly at the different measuring levels. Two or three weeks after the driving the rate of the load increase slowed down. After about eight weeks the load increase was very small at the upper levels.

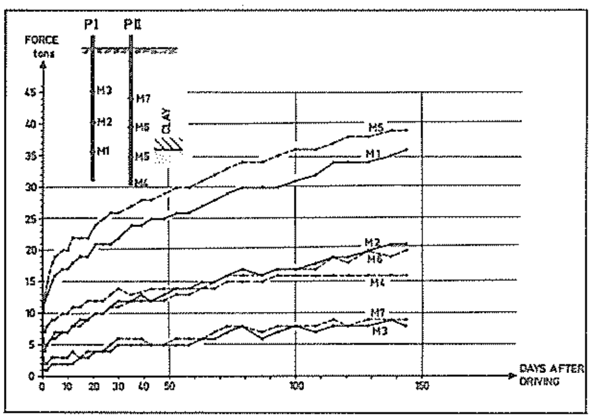

Fig. 6 Recorded forces in piles PI and PII

In Fig. 7 the measured loads have been plotted at different time intervals after the driving. The dotted line in this figure represents the weight of the pile. The line marked 0 represents the load distribution immediately after the driving. It can be seen from Fig. 7 that the axial load in both piles was less than the weight of the pile immediately after the driving down to a depth of nearly 40 m. The load in the piles increased with time and 144 days after the driving, the pile load exceeded the weight of the pile by 25 to 30 tons at the level of the interface between the clay layers and the underlaying silt and clay layer. This load is still increasing five months after the driving. Gauge M 4 in pile PII indicates that most of the load due to negative skin friction along the pile is resisted as positive skin friction in the underlaying silt and sand layers. The average skin friction can be calculated from the load distributions. It is proportional to the load difference between adjacent levels. In Fig. 8 is shown the average skin friction resistance for the two piles. The points which on the same occasion represent the average skin friction have been connected. The measurements indicate that the skin friction resistance increases approximately linearly from zero at the ground surface to 1.4 t/m^2 at a depth of 40 m. The skin friction resistance below this depth was positive. The negative skin friction corresponds to 17 % of the undrained shear strength of the clay or to 5 % of the effective overburden pressure of the soil.

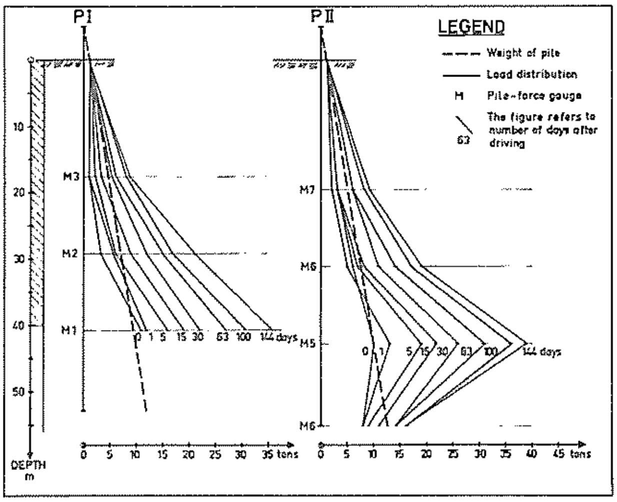

Fig 7 Vertical distribution of load in the piles at different times after driving

The recorded bending moments increased after the driving. However the increase was small. The bending moment at gauge M5 which immediately after the driving recorded 3.2 tonm increased by 12 % to 3.6 tonm five months after the driving. This relative increase was the largest recorded by any of the gauges.

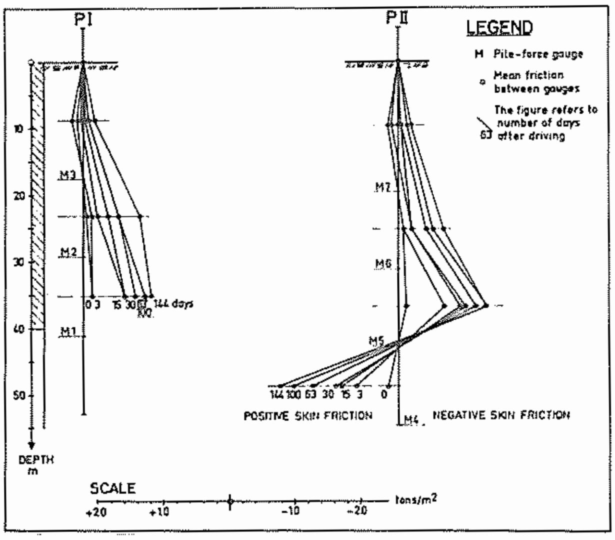

Fig. 8 Calculated average skin friction at different times after driving

CONCLUSIONS

The test results indicate that negative skin friction can be caused by the remoulding of the clay around driven piles and the subsequent reconsolidation of the soil even if the settlements of the soil are very small. The measured negative skin friction after a period of five months corresponded to 17 % of the average undrained shear strength of the clay or to 5 % of the average effective overburden pressure. The resulting axial forces in the piles were resisted by positive friction in the silt and sand layers at the lower parts of the piles. Considerably higher values of the skin friction resistance will undoubtly develop when a fill is placed over the area.

ACKNOWLEDGEMENTS

This project, which is carried out in co-operation with the A. Johnson Institute for Industrial Research, has been supported financially by the National Council of Building Research (Statens Råd för Byggnadsforskning).

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Contribution to the discussion in the Specialty Session on Negative Skin Friction and Settlements of Piled Foundations.

Negative Skin Friction for Long Piles Driven in Clay

by Bengt H. Fellenius, Swedish Geotechnical Institute, Stockholm, Sweden

The aim of this discussion is to inform about further developments of the test program that is presented in the paper "Negative Skin Friction for Long Piles Driven in Clay" by myself and Dr. Bengt B. Broms.

The paper shows that driving of displacement type piles in clay causes remoulding of the surrounding clay and excess pore pressures. The excess pore pressures dissipated after about 150 days. The recorded further settlements 150 days after the pile driving were within the accuracy of the settlement gauges, i.e. 1 mm.

The loads in the piles increased almost linearily with time (Fig. 1). There is, however, a slight decrease of the loading rate. The loads at the bottom of the clay layer were 52 and 56 tons in piles P I and P II, respectively, 420 days after the installation. Fig. 2 shows the load distribution in the piles. Gauge M 4 in pile P II indicates that the load is resisted by positive friction in the silt and sand layers below the clay. Of the 46 tons, which has been the load increase since the installation, only 10 tons reached the pile tip.

The negative friction after 420 days corresponds to 28 % of the average original undrained shear strength of the clay or to 9 % of the average effective overburden pressure.

In the next phase of this investigation, which will begin shortly, a dead load of 80 tons will be placed on each test pile. In phase three (after further six or ten months) a 2 m high gravel fill will be placed around the two test piles over an area 40 by 40 meters. The results of the various test phases will be published by the Swedish Geotechnical Institute.

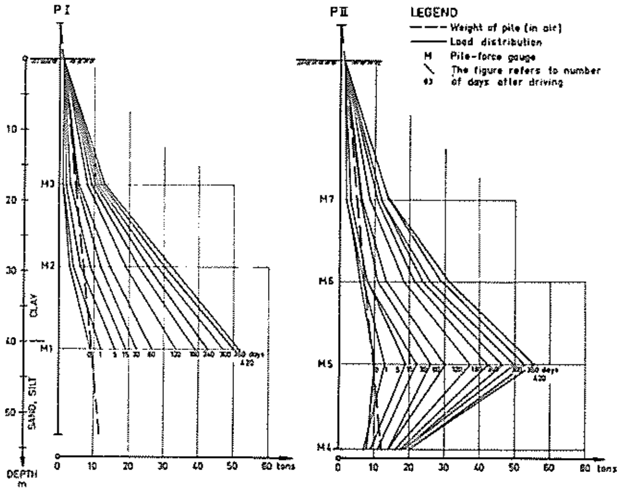

Fig. 2 Vertical distribution of load at different times after driving

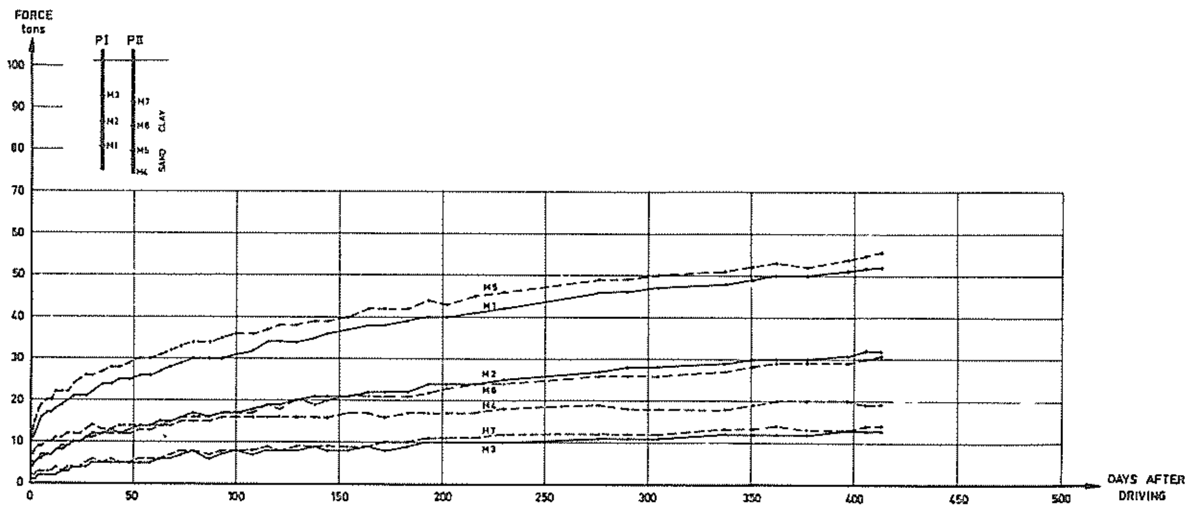

Fig. 1 Recorded loads in piles P I and P II

Contribution to the discussion in the Specialty Session on Negative Skin Friction and Settlements of Piled Foundations.

Design of Pile Groups with Respect to Negative Skin Friction

by Bengt B. Broms, Swedish Geotechnical Institute, Stockholm, Sweden

This review is concerned with the methods which are used in Sweden to design pile groups with respect to negative skin friction. Measurements have shown that drag forces (negative skin friction) develop along single piles or pile groups when the soil settles with respect to the piles. The settlement of the soil may be caused by the placement of a fill around the piles, a lowering of the ground water table or by remolding of the soil during the driving. The load increase due to negative skin friction often approaches the allowable load on the pile when the pile length exceeds 20 to 30 m. The negative skin friction can be large even if the settlements are small (a few millimeters). Three different cases are generally recognized as illustrated in Figs. 1a, 1b and 1c.

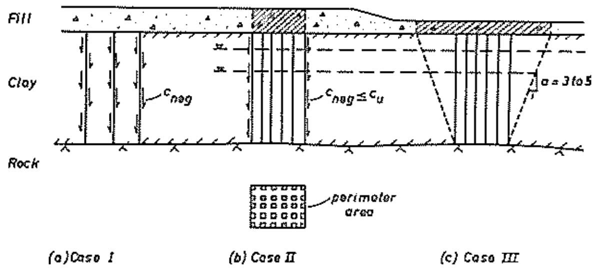

Fig. 1 Calculation of negative skin friction

Case 1. The piles in the pile group shown in Fig 1a are assumed to be widely spaced. The height of the fill placed around the pile group or the lowering of the ground water table is large enough to develop the maximum skin friction along all the piles in the pile group. It is assumed furthermore that the piles are point bearing and that the pile points do not move when the load in the piles is increased by negative skin friction. This is generally the case in Sweden where most piles are driven to rock. The neutral point is often assumed to be located close to the pile point since the compression of the pile is generally neglected. The negative skin friction resistance will be overestimated by this method if the settlements are small or the pile points move.

There are two different methods to calculate the negative skin friction which is assumed to act along the full length of the pile. The first method is generally used when the liquid limit of the soil is less than about 50. The negative skin friction c_{neg} is according to this method calculated from the equation $c_{neg} = a\,\sigma_v^1$ where "a" is a coefficient and σ_v^1 is the effective overburden pressure including the weight of the fill. In this case it is thus assumed that negative skin friction is proportional to the effective overburden pressure σ_v^1. The values of the coefficient "a" which vary between 0.2 to 0.3 are based on test results reported by Johannesen and Bjerrum (1965). These values have been derived from pile load tests in a clay with a relatively low c/p-ratio and a low liquid limit.

In the second method, which is often used when the liquid limit of the clay exceeds approximately 50, it is assumed that the negative skin friction c_{neg}, is equal to the adhesion of the surrounding soil with respect to the pile surface. The values which are used to calculate the negative skin friction are generally the same as those which are used to calculate the bearing capacity of single piles in clay as indicated by the following table.

Table I. Evaluation of the unit skin friction resistance c_a

(a) $c_u < 0.5$ kp/cm^2	Skin-friction resistance, c_a
Steel piles	0.5 c_u
Concrete piles	0.8 c_u
Timber piles	1.0 c_u
(b) $c_u > 0.5$ kp/cm^2	
Steel piles	0.1 kp/cm^2
Concrete piles	0.3 "
Timber piles	0.5 "

One may question if the same values of the unit adhesions should be used to calculate the negative skin friction as those used in the calculations of the bearing capacity. The maximum value of the adhesion governs the negative skin-friction resistance while the minimum value is of interest when the bearing capacity is calculated. It may be argued that all piles which are affected by negative skin friction should be designed to resist a drag load which corresponds to the full undrained shear strength of the soil as measured by e.g. vane tests, unconfined compression tests or fall-cone tests instead of a reduced value which corresponds to a lower limit of the adhesion.

Case II. It is assumed in Fig 1b that the piles are closely spaced and that the height of the fill or the lowering of the ground water table are sufficiently large to mobilize the full skin friction resistance or the full shear strength of the soil along the perimeter area of the pile group. The average skin friction c_{neg} probably will be less than the undrained shear strength of the soil. The perimeter of the pile group partly follows the pile surfaces. The undrained shear strength of the soil is generally used for the perimeter area which follows the undisturbed soil while reduced values are used for the area which follows the surfaces of the piles. The pile group carries also the weight of the fill which is located immediately above the piles or the increase of the boyant weight of the soil within the pile group when the ground water table has been lowered. It is thus assumed in the calculations that the vertical effective stress in the soil within the pile group will not at the level of the rock surface be affected by a lowering of the ground water table. The total drag force is assumed to be equal to the sum of the negative skin friction along the perimeter area and the weight of the fill within the pile group. This total drag force is distributed equally among all the piles in the pile group. It may, however, be argued that the change of the boyant weight of the soil within the pile group should not be included in the calculations since the total weight of the soil is not affected. This is frequently done by many designers.

Case III. The piles in Fig 1c are relatively closely spaced and the height of the fill or the lowering of the ground water table is moderate to small. In this case it is assumed that the pile group carries a load which corresponds to the weight of the fill within the area indicated in Fig 1c. The sides of the inverted truncated soil cone have a slope of 1:3 to 1:5. The proceedures used by different designers differ due to lack of test data. If settlements are caused by a lowering of the ground water table it is often assumed that the total drag force corresponds to the change of the boyant weight of the soil below the area indicated in Fig 1c. The pile group will in this case carry a load which is equal to the weight of a column of water with a height equal to a lowering of the ground water table.

It should be pointed out that the available test data are extremely limited, that measurements primarily have been made on single piles and that the settlements which have occured around the instrumented piles have been relatively large. When additional test data become available it is likely that the presently (1969) used design methods will be changed. It should, however, also be pointed out that these design methods generally do not take into account the increase of negative skin friction which is caused by remoulding of the soil during driving. This increase of the pile loads can be very large in Cases II and III. For these cases the calculated negative skin friction is normally moderate.

There are several methods to calculate the allowable load for a pile group allow as indicated below.

$$Q_{allow}^{group} = \frac{Q_{ult}^{group} - Q_{neg}}{F} \qquad (1)$$

$$Q_{allow}^{group} = \frac{Q_{ult}^{group}}{F} - Q_{neg} \qquad (2)$$

In the first case the safety factor F is applied on both the calculated ultimate bearing capacity of the pile group as well as on the negative skin friction Q_{neg}. In the second case the factor of safety is only applied on the calculated ultimate capacity. The first method which is normally used in Sweden gives a higher allowable load than the second method.

The consequenses of exceeding the ultimate capacity of the pile group are often not serious since excessive pile loads caused by the negative skin friction will generally not cause collapse of the supported structure. If a pile group settles then the negative skin friction changes into positive skin friction which will contribute to the bearing capacity of the pile group. Thus a relatively low safety factor can be used with respect to negative skin friction.

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NEGATIVE SKIN FRICTION ON PILES IN CLAY – A LITERATURE SURVEY

Bengt H Fellenius, Swedish Geotechnical Institute, Stockholm, Sweden

Introduction

In 1965 the Axel Johnson Institute for Industrial Research in cooperation with the Swedish Geotechnical Institute initiated an investigation on negative skin friction on piles in clay. The work is supported financially by the Swedish Council for Building Research.

The first step in this investigation was a literature survey. This article is a survey of papers, which have been published on negative skin friction in clay up to the end of 1968.

Contents

1. Statement of the problem

When the soil around a pile settles, shear forces are induced along the pile surface and the load in the pile is increased. Usually this phenomenon is called negative skin friction. In cohesive materials the terms "negative skin cohesion" and "negative skin adhesion" are sometimes used.

2. Previous investigations

Under this heading some cases, where negative skin friction has caused structural damage (2.1), are described as well as cases where negative skin friction has been taken into account in the design (2.2). Also the research on drag forces in piles is reviewed (2.3).

2.1 Structural damage caused by drag forces

McKay (1938)[x)] describes how a fill around a building which had been founded on concrete piles, caused large settlements around the piles. The piles were not designed to carry the resulting drag forces and the piles failed. No calculations were given by McKay. McKay also describes how piles for a river quay in 10 m depth of water were damaged, by drag forces caused by a fill of a silty material, which had been deposited around the piles. Negative skin friction developed both in the silty fill and in the underlying materials through which the piles had been driven.

Florentin and l'Heriteau (1948) describe a case where concrete piles with 0.8 m diameter were driven through a 4 m thick fill and through underlying layers of very soft clay into a dense sand sublayer. The piles settled 30 cm in three years, which according to Florentin and l'Heriteau "can only be explained by negative skin friction on the piles". The settlement of the piles was indirectly caused by unsatisfactory pile driving. The settlement stopped, when the pile tips reached the underlying dense sand.

Moore (1949) describes a case where a wooden pile had been driven through a stony fill and underlying compressible soils down to firm layer. The thickness of the fill was 13 m and the compressible soils

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x) See literature index in section 5.

consisted of a 9 m thick layer of mud and 6 m of clay. The settlements of the fill and of the compressible layers caused drag forces of such magnitude that the pile pulled loose from the supported structure. The design pile load had been determined from a dynamic pile formula. (The pile might have been damaged during the driving.) The drag forces as estimated by Moore were 170 tons for the fill and 12 tons for the mud and clay.

Milner (1957) describes a tunnel, which was founded on 40 cm diameter cast-in-place concrete piles. The piles were driven through 8 m of soft clay into an underlying layer of mica. An 8 m thick fill caused settlements in the clay which resulted in estimated drag forces of 80 tons. Because of this additional load the piles were pushed down into the mica and the tunnel walls cracked.

Kapp (1965) relates in a letter that settlements of about 2 m caused drag forces of such magnitude around unreinforced concrete piles under the floor of a hangar building at La Guardia Airport, New York, that some piles were pulled loose from the floor. Attempts were made at La Guardia to measure the drag force by measuring the force that was required to prevent the pile from sinking. When this force reached 30 tons, the pile failed in tension.

Further examples are given by Chellis (1951).

2.2 Cases where drag forces have been considered in the design

Golder and Spence (1960) mention that piles for a bridge, where large drag forces were expected, were covered with half an inch thick layer of asphalt. The piles were put down in predrilled holes in the ground to protect the asphalt.

Reséndiz (1964 a) describes a case where concrete piles of variable diameter were used; one upper part (Ø 35 cm of about 30 m length) and one lower part (Ø 20 cm of about 3 m length). The piles were driven to a firm sand layer. The allowable pile load was the estimated ultimate bearing capacity of the pile tip, i. e. the safety factor against soil failure was equal to 1. When drag forces on the piles were caused by the settlement of the ground, the pile tips were pressed down into the sand layer and thus the building on the piles settled at the rate of the ground surface. The tests have been described in detail by Reséndiz (1964 b).

Girault (1964) describes the difficult pile foundation problems in Mexico City. These problems are caused by the lowering of the ground water level and the resulting large regional settlements. Buildings which are founded on end-bearing piles appear to rise with respect to the ground and buildings on friction piles settle unevenly, because of large drag forces on the piles. Girault has proposed a combination of friction piles and end bearing piles (See fig. 2.2.1). When the friction piles (A) sink, load is transfered through the soil between section α - α and section β - β on to the end bearing piles (B). A building founded on this piling system will, according to Girault, settle evenly at the same rate as the ground surface.

Further examples can be studied in papers by Zeevaert (1957 a), (1957 b) and (1963, Correa (1961) and Locher (1965).

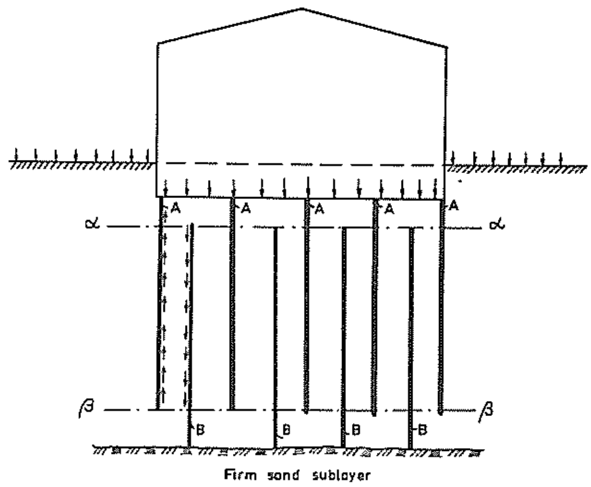

Fig. 2.2.1 Combination of friction and end bearing piles. A = friction pile. B = end bearing pile.

2.3 Research on drag forces on piles

Gant, Stephens and Moulton (1958) describe tests on steel pipe piles which were equipped with electrical strain gauges. The purpose of the tests was to determine the allowable pile load for a bridge foundation. The soil consisted of 7 m of sandy fill, 10 m of silt and a sublayer of sand. The piles were of 30 m length and were thus driven about 12 m down into the sand. Drag loads of about 90 tons were measured four months after the driving of the piles.

Tchebotarioff (1958) in the discussion following the article by Gant et al., indicated a few possible errors in the measurements and pointed out that only a part of the measured axial pile forces could have been caused by drag forces.

Weele (1964) describes a case where the drag forces acting along wooden piles of different lengths have been determined. Three piles were driven at a spacing of 4 m down to a depth of 13.3 m (No 1), 15.5 m (No 2) and 17.5 m (No 3). The soil consisted of 6 m of sand, 7 m of clay and a sand sublayer, which at a depth of 17 m was very firm. The ground water level in the area was initially 1 m below ground surface. In an initial phase the ground water level was lowered 6 m and in a second phase an additional 3 m. This caused settlements in the soil and the piles were dragged down. See fig. 2.3.1.

Along the upper part of the piles the settlement in the soil was larger than the settlement of the piles and drag forces were developed along this part of the piles. In the lower part the settlements were smaller than those of the piles and thus positive skin friction was obtained along the surface of the piles.

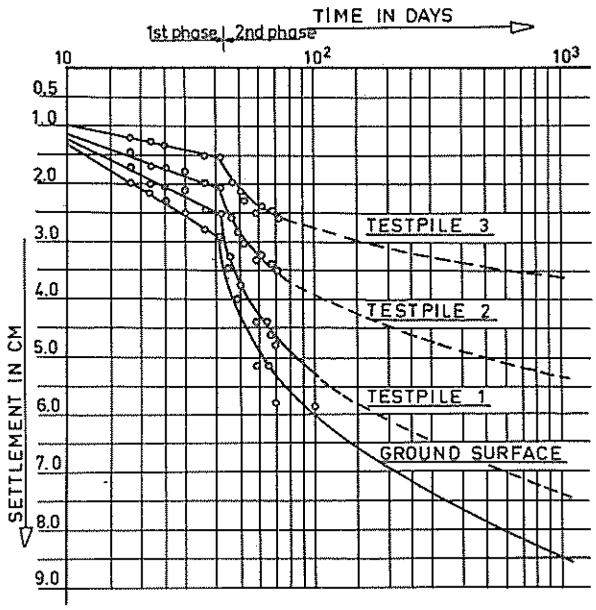

Fig. 2.3.1. Settlement-time relation for the ground surface and the 3 test piles during the two stages of the ground-water lowering. (After Weele, 1964).

From pulling tests on concrete piles the average shear stress along the pile surface were determined as a function of the pile displacement. The maximum resistance was obtained, at a displacement of about 10 mm. The magnitude varied for the three types of soil:

In the upper sand layer	3 t/m^2
In the clay	1.5 t/m^2
In the sand under the clay	20 t/m^2

From these results the drag forces on the piles were calculated. The total drag load on pile No 1 was estimated to 24 tons.

2.4 Comments on the reviewed cases

Drag forces on piles can be large and cause piles to settle. This may damage the supported structures. However, in the literature on drag forces on piles, no reference has been found of the influence of negative skin friction on piles which have been driven to a very firm layer. Severinsson (1965) reportes that no settlement has been observed for 300 structures in Sweden founded on long, high quality, precast concrete piles driven to rock or to similar layers through settling soil layers.

The magnitude of the negative skin friction given by Weele may be compared with the values recommended by Tomlinson (1963) for the calculation of drag forces along piles.

Clay and silt	0.7 - 3 t/m^2
Very stiff clay	5 - 20 t/m^2
Loose sand	1 - 4 t/m^2
Dense sand	3 - 7 t/m^2

In the same paper Tomlinson mentiones that drag forces can be caused by the remoulding and heave of sensitive clays during pile driving, as is shown in Fig. 2.4.1. This effect has also been discussed by, among others, Casagrande (1932), Cummings et. al. (1948), Tschebotarioff (1949) and Zeevaert (1949).

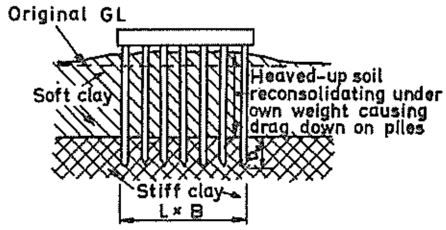

Fig. 2.4.1 Effects of ground heave on groups of driven piles in clay

3. Calculation of drag forces

Under this heading the existing theories for determining drag forces are reviewed.

3.1 Terzaghi and Peck (1948) recommend the following method for determining drag forces on pile groups. The soil around the piles consists of a compressible layer of clay and a surcharge causes the clay layer to settle. The axial force (F) on a pile group is then the sum of the structural load (F_Q) and the drag force (F_n). The drag force itself is equal to the sum of the weight of the fill within the pile group (F'_n) and the skin friction (F''_n) along the perimeter of the pile group. F''_n is divided equally between the piles. F''_n can, according to Terzaghi and Peck, vary between zero and the maximum value calculated from equation 3.1.3.

$$F = F_Q + F'_n + F''_n \quad ^{x)} \qquad 3.1.1$$

F_Q = Structional load on the pile

F'_n = Weight of fill carried by one pile

$$F'_n = \frac{A}{n} \cdot \gamma \cdot H_F \qquad 3.1.2$$

A = Area of the pile group
n = Number of piles
H_F = Thickness of fill
γ = Unit weight of the surcharge material

F''_n = load caused by settling of the surrounding clay

$$F''_n \leq \frac{c}{n} \cdot H_L \cdot \tau \qquad 3.1.3$$

c = Circumference of pile group
H_L = Thickness of clay layer
τ = Average shear strength of clay

To reduce of the magnitude of the negative skin friction Terzaghi and Peck recommend that the distance between the piles should not exceed 2.5 pile diameters.

3.2 Zeevaert (1959) states that the ultimate point bearing capacity of piles, which have been driven through compressible soil to firm sand layers, can be calculated from the equation

$$F_B = N_r \cdot a \cdot p_{oh} \qquad 3.2.1$$

F_B = Failure load (soil failure)

N_r = A factor which is a function of the angle of internal friction (φ) and the relative density of the sand (D_r).

$$N_r = N_q \cdot (D_r + 0.1)$$

Nq is a function of φ. It is assumed that the shearing strength is fully mobilized on the surface of failure.

a = Pile area

p_{oh} = Effective vertical soil pressure in the sand at the pile tip.

The failure load, F_B, depends on the effective overburden soil pressure at the pile tip. When the soil settles, the vertical pressure, p_{oh}, is reduced to p_{vh} by load being transfered to the pile. (Fig. 3.2.1). Thus the failure load, F_B, is reduced to F'_B.

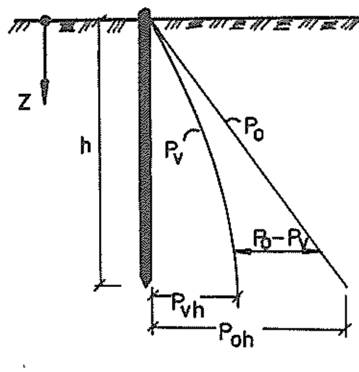

Fig. 3.2.1
Reduction of the effective vertical pressure

The shear force in the soil along the pile is, according to Zeevaert, a function of p_o

$$s = K_o \cdot \tan\varphi \cdot p_o \qquad 3.2.2$$

K_o is the ratio between the horizontal and the vertical pressure in the soil at the pile surface and

$\tan\varphi$ is the ratio between the horizontal pressure against the pile and the shear force along the pile.

When p_o is reduced to p_v, the shear strength is also reduced. This is a basic assumption for Zeevaert's mathematical deriving of the following formulas. The total drag force is derived at

$$F_n = \frac{1}{n'} \cdot (\bar{\gamma} h - p_{vh}) \qquad 3.2.3$$

where n' = Number of piles per unit area

$$p_v = \frac{\bar{\gamma}}{m} (1 - e^{-mz}) \qquad 3.2.4$$

$$p_{vh} = \frac{\bar{\gamma}}{m} (1 - e^{-mh}) \qquad 3.2.5$$

$$m = n' \, U K_o \tan\varphi \qquad 3.2.6$$

This approach to negative skin friction is mainly dependant on the validity of Eq. (3.2.1) and (3.2.2). These equations do on some occasions not agree with experimental facts as shown for example by Vesic (1964). Furthermore, it is doubtful whether Eq. 3.2.2 is valid in clay.

Zeevaert's approach refers to large pile groups of piles. The equations can not be used for a single pile without making certain assumptions about the load distribution around the pile.

de Beer (1967) has developed Zeevaert's approach and has published nomograms that simplify the use of the equations. In the case of a surcharge on the ground surface de Beer assumes that the area affecting the single pile is equal to the base area a 90° cone with the height equal to the depth of the settling layer.

3.3 Buisson, Ahu and Habib (1960) have studied theoretically the stress distribution along a pile which has been driven into a compressible clay. Settlements of the compressible layer are caused by a fill placed on the ground surface. The pile tip rests on sand.

The following equation is given (See Fig. 3.3.1):

$$F_{point} = F_Q + F'_n + \tau \cdot U \cdot Z - \tau \cdot U \cdot (H-Z) \qquad 3.3.1$$

where F_Q, F'_n and F_{point} are defined in Fig. 3.3.1

$$F'_n = 0.30 \cdot \frac{\gamma \cdot H_F^2}{2} \cdot U \qquad 3.3.2$$

the term 0.30 is a "friction coefficient", which can be compared with $K_o \cdot \tan\varphi$

γ = unit weight of fill

U = perimeter of pile

τ = shear strength of clay

Z = depth to the neutral point, i.e. the depth along which the drag forces act.

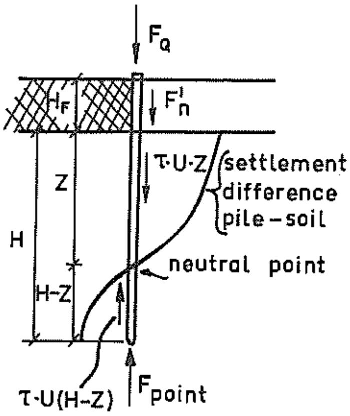

Fig. 3.3.1

The distance, Z, to the neutral point can be determined by calculating the pile compression and the soil compression separately by iteration procedure. The tip of a pile which has been driven to bedrock or to a similar very firm layer, does not move when loaded. The neutral point must in this case be very close to the pile tip due to the low compressibility of the pile. Thus the last term in the equation (3.3.1) is small and can be neglected.

3.4 Elmasry (1963) has tested 80 cm long model steel piles, which were provided with strain gauges. The piles were driven through a 50 cm thick layer of silty clay, which was mixed in a soil mixer before it was placed in the test cylinder. The pile tips were located in a sand layer. A load was applied on the clay surface through a plate. The forces in the model piles were measured for different loading conditions. Elmasry derived from the test results an equation to calculate the drag force. The maximum value of the drag force was found to be proportional to

the consolidation pressure from the surface load (p_c),

the pile perimeter (U),

the thickness of the clay layer (H),

the ratio between the dry weight of soil per horizontal unit area and the consolidation pressure

$$\lambda = \frac{\gamma_d \cdot H}{p_c} \quad \text{and}$$

the natural water content (w).

The maximum value of the drag force was a linear function of these parameters taken separately. Combining these functions Elmasry derived the following formula.

$$F_n = F_{n-s} + 0.416 \cdot (2 \cdot p_c \cdot H \cdot U - 0.70 \cdot \lambda^2 \cdot \frac{w \cdot p_c^3}{\gamma_s \cdot \gamma_d}) \qquad 3.4.1$$

F_{n-s} is the drag force due to the consolidation of the clay under its own weight. This drag force can be calculated from the following equation.

$$F_{n-s} = \int_0^H U \cdot \tau_{p-s} \cdot dH = U \cdot H \cdot \tau_{p-s_{mean}}$$

$\tau_{p-s_{mean}}$ is given as 30 % of the shear strength of the clay at the final stage of consolidation. Elmasry referes to Zeevaert (1957 a).

3.5 Johannessen and Bjerrum (1965) describe results from tests on two steel piles, 47 cm in diameter, with a length of about 60 m, which were driven to bedrock through a layer of clay of about 50 m thickness.

After the piles were installed a fill with a height of about 10 m was placed around the piles. A slide in the underlying clay disturbed the tests and results were obtained from only one of the two test piles. The deformations caused by drag forces were measured on this pile through a system of rods. Since the modulus of elasticity of the steel material was known, the average shear stress, s, along the pile surface could be calculated. The shear stress corresponded to the equation $s = p \cdot K_o \cdot \tan\varphi$.

The measured drag load at the pile tip exceeded the ultimate bearing capacity of the pile tip.

The tip penetrated 6 cm into the underlying rock. This penetration occured slowly and in short steps. At each increment the pile was partly unloaded. The drag force decreased momentarily.

The largest calculated value of $K_o \cdot \tan\varphi$ was 0.20, which corresponded to a total pile shortening of 2.5 cm. The calculated total maximum drag load was 300 tons. On the basis of these results Bjerrum and Johannessen recommend that the calculated drag loads on piles in loose marine clays should correspond to 20 % of the final effektive vertical pressure in the clay, i.e.:

$$K_o \cdot \tan\varphi = 0.20$$

$$F_n = \int_0^H U \cdot s \cdot dz = 0.20 \int_0^H U \cdot p \cdot dz \qquad 3.5.1$$

3.6 Brinch Hansen (1968) has investigated theoretically the skin friction on piles in general and applied the results on negative skin friction. Brinch Hansen derived the following equation

$$s \leq \frac{p}{4+6.8\ D/b} \qquad 3.6.1$$

s = shear force (negative)

p = surface load per unit area and/or pore pressure decrease in the soil

D = pile diameter

b = distance between piles in a group

Equation (3.6.1) gives an upper limit of the negative skin friction. Brinch Hansen states that the maximum negative friction is also governed by the maximum pull out resistance of the pile. The theoretical expressions for the pull out resistance are given in the paper.

Brinch Hansen applied Eq. (3.6.1) on the pile tests reported by Johannessen and Bjerrum (Chapter 3.5 above). An extremely good agreement was obtained between measured and calculated drag load. This agreement may only be a coincident as is pointed out in the paper.

3.7 Other studies

Other theoretical studies of negative skin friction have been published by Yokowo et. al. (1968) and Poulos et. al. (1968). It is assumed in these studies, however, that both the soil and the pile material are purely elastic. These assumptions limit the practical application of the theories as soil and especially the soft clays in Sweden do not behave elastically. These studies have therefore not been cited here.

4. Comments and conclusion

Certain assumptions and extensive simplifications must be made as to the soil conditions when applying the equations by Buisson et. al. and Elmasry on practical cases. Since the calculated drag forces are to a large extent dependent of these assumptions, it is difficult to estimate the accuracy of the proposed methods.

When the negative skin friction is calculated according to the method proposed by Terzaghi and Peck, it is difficult to choose the value of to be used in the calculation of F_n''. It is far from certain that shear is fully developed along the full perimeter area of a pile or a pile group. The value of τ depends also on the remoulding of the soil during the pile driving and on the resulting reconsolidation of the soil.

In Zeevaert's theory the shear force along the pile depends on the resulting overburden pressure in the soil after unloading. Part of the overburden is transferred to the pile and will depend on the coefficient $K_o \cdot \tan\varphi$. These factors vary with the soil conditions and the method of pile installation. However, very little is known about how negative skin friction is affected by these factors.

Johannessen and Bjerrum base their recommendation on the results from a single test pile in loose marine clays in Norway. Their equation (Eq. 3.5.1) is simple and can easily be applied on practical cases. As mentioned above, the general validity of this equation remains to be proved. However, the equation is the only one that is derived from an actual field test.

Before any definite recommendations can be given the parameters which affect the skin friction such as soil conditions, soil characteristics and magnitude of the consolidating pressure must be evaluated in the field and in laboratory. Especially full scale tests should be of value. Furthermore, it will be essential to investigate the effects of pile driving and the rate of soil settlement upon the magnitude of the shear forces.

Once the maximum drag load can be evaluated the important question of how this drag load affects the behaviour of the pile and how it can be taken into account when designing the actual pile foundation remains to be answered. Fellenius (1964) has proposed for a point bearing concrete pile driven in underconsolidated clay (in southern Sweden) that a drag force, corresponding to the original undrained shear strength of the clay, should be assumed to act along the pile surface down to a maximum depth of 20 m, or for piles shorter than 40 m, only along half the length of the pile. The sum of the drag load calculated in this way, the maximum design load and the weight of any fill located within a pile group, should not exceed the ultimate strength of the pile section, i.e. about 0.65 % of the cube strength.

This literature survey indicates that drag forces can cause piles to sink with fatal results, when the piles are supported by sand strata or any material in which the piles can penetrate. Local experience will on many occasions indicate whether a proposed design will be safe or not. When high design loads are used or the driving conditions are severe, a more thorough penetration of the problems will be required.

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7. Symbols

General

V volume

W weight

G_s safety factor

Stress and strain

p effective vertical soil pressure

p_o original effective vertical soil pressure

p_v decreased effective vertical soil pressure

p_c consolidating pressure caused by overburden

s shear force

τ shear strength

τ_s shear strength after consolidation

$\tau_{p-s} = 0.30 \cdot \tau_s$

Soil classification

w water content in % of dry weight

γ unit weight

$\bar{\gamma}$ effective unit weight

γ_s unit weight of solid particles

γ_d dry unit weight

φ angle of internal friction

Dimensions

B width

L length

H depth, thickness of soil layer

z depth coordinate. A certain value of z writes h.

Pile symbols

D pile diameter (side)

U pile perimeter

a pile area

Z depth to the neutral point

F axial force in pile

F_Q structure load

F_B failure load

F_n drag force

- F'_n drag force in or from fill
- F''_n drag force caused by soil settling around a pile

C perimeter of the polyhedron surrounding a pile group

A area of the polyhedron surrounding a pile group or the fill area affecting the pile

n number of piles in a pile group

n' number of piles in a pile group per unit area

K_o the ratio between the vertical and the horizontal soil pressure

$m = n' \cdot U \cdot K_o \tan\varphi$ acc. to eq. 3.2.6

NUCLEAR RADIATION IN CONSTRUCTION CONTROL OF EARTH AND ROCKFILL DAMS

RADIATIONS NUCLEAIRES DANS LE CONTROLE DE LA CONSTRUCTION DES BARRAGES

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Swedish State Power Board, Stockholm, Sweden.

SYNOPSIS

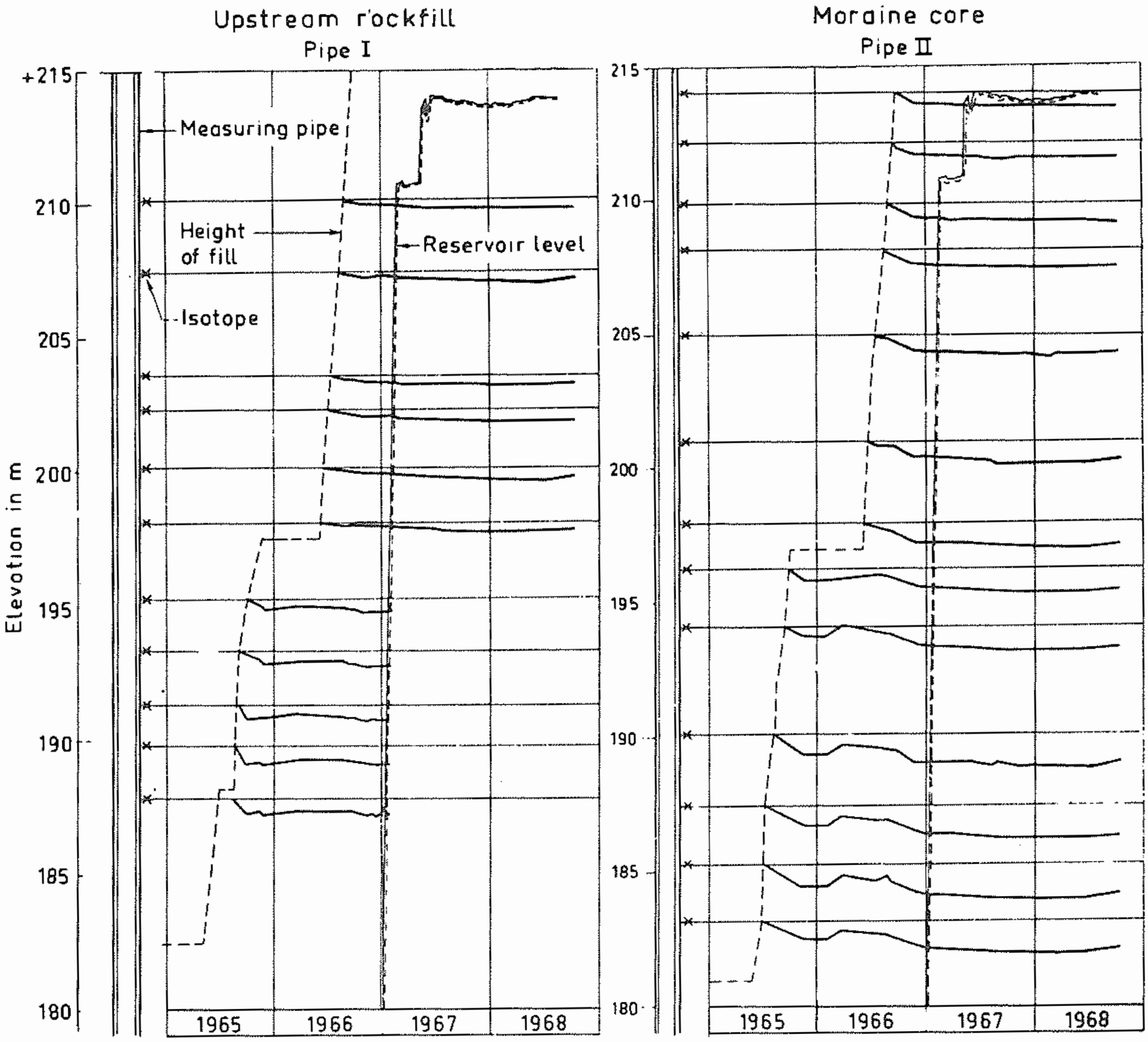
During the later years the nuclear radiation technique has been widely used in soil investigation in order to determine certain characteristic properties of soils. Principally the development in this field has been concentrated on methods for determining the moisture content in soils and the density conditions in situ and in compacted embankments, whereas very little attention has been devoted to the possibility to apply the new technique to other fields of soil- and rock mechanics (Bernell and Lindbo, 1965). In connection with the design and construction of several earth and rockfill dams the Swedish State Power Board has developed the nuclear radiation technique for solving many problems in the construction control. The experiences thus obtained have resulted in reliable and simple methods for determining settlements in earth and rockfill embankments and for controlling the effect of grouting in soil and rock. These applications of the nuclear method are briefly described in the following.

USE OF NUCLEAR METHODS FOR MEASURING SETTLEMENTS

For determining the vertical movements in compacted earth and rockfills the telescoping pipe device with the horizontal crossarms is often used. This measuring method has also been applied to Swedish dams, but experience has shown that it has many disadvantages (Bernell 1964). In embankments, which have been filled according to the wet-compaction method, the horizontal movements often become so large that the pipes break off. Another draw-back is that the pipes and the crossarms must be installed during the construction periods and thus frequently cause trouble during the filling operations.

In most of the dams, constructed by the State Power Board as well as by other power companies in Sweden during the later years, the nuclear method was chosen for determining the simultaneous settlements at different elevations in the fills (Bernell, 1967). According to this method a radioactive isotope Co^{60} is placed in the measuring point and the radiation from the isotope is determined by means of a Geiger-Müller detector. Generally the measurements are carried out in a steel pipe, protected from the surrounding soil by an elastic pipe casing. The location of the isotope is indicated by a peak value in the radiation intensity and thus the elevation of the measuring point can be determined.

Experience has shown that the nuclear method can easily be applied to settlement measurements in both soil and rockfill embankments. Generally the isotopes are placed in the fill close to the pipe and at different elevations Fig. 1 shows a typical diagram from settlement measurements in an earth and rockfill dam. In this case the measuring section included both the core and the rockfill zones Long term measurements in this and many other dams have proved that the method is very simple and reliable. The accuracy in the results is also very good, about 1 cm at a depth of 100 metres.



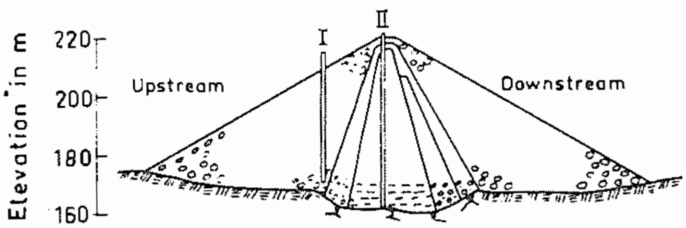


Fig. 1 Settlements in an earth and rockfill dam determined by means of nuclear measurements

Since the installation and the maintenance of the pipes often cause difficulties with regard to the filling works, a modified method for performing the measurements has successfully been applied. The method is based on the fact that the range for the radiation in a soil is relatively large also for very small quantities of the isotope. In a recently completed dam small quantities of Co^{60} were placed directly on the compacted surface within an area of 2 x 2 metres and with a spacing of 0,25 metres. After completion of the filling works, measuring holes were drilled down to the surface with the isotopes. Although the holes had to be drilled to a depth of about 30 metres, the measurements could be successfully executed.

The nuclear method is also of great value for determining additional settlements in natural soil strata beneath supporting foundations.

In this case the isotope is placed in a mechanical shot-device, which is inserted in the pipe after drilling of the hole. By means of the shot-device the isotope can be placed in the soil at any desired depth. This method can be applied to many cases in foundation engineering, where it may be of special interest to obtain reliable data concerning the compressibility and the settlement conditions in the underground.

In construction control it is often desirable to arrange the measurements in such a way that both the vertical and the horizontal movements can be determined by using the same installations. For measuring horizontal movements the strain gauge inclinometer is generally used (Kallstenius and Bergau, 1961). By choosing the diameter of the pipe so that the pendelum can be inserted in it, the same pipe can be used for both vertical and horizontal measurements. This combination of measurements has successfully been applied to the control of the stability of slopes.

USE OF NUCLEAR RADIATION IN GROUTING CONTROL

Most of the dams, constructed in Sweden during the later years, are situated within the mountain region in the northern part of Sweden. The soils in this area commonly consist of moraine deposits from the last ice age. Because of geological processes the composition of these deposits is very irregular and generally they contain both impervious moraines and lenses of pervious sediments. Therefore, construction of dams in this region often involves difficult foundation problems, especially in cases where the dam has to be founded on partly permeable deposits.

Because of the erratic soil conditions, foundation grouting in the dam site is almost always needed in order to reduce the underseepage after the rise of the reservoir level. But very often it may be a major problem during the construction stage to decide the extension of the grouting works. Sometimes the need for grouting becomes apparent only after the reservoir has been filled and in such cases the foundation must be grouted through the dam embankment. This method has certain advantages, but evidently the grouting operation must be carefully controlled because of the risk of cracking and eroding the soil in the foundation. In order to eliminate this risk the State Power Board has applied the nuclear radiation method to the control of grouting operations. The method is based on investigations of the radiation in soils, compacted to various porosities.

At a recently completed dam, founded on natural moraine deposits, leakage occurred under the foundation after the rise of the reservoir level. The sealing measures included the performance of a grout curtain, extending through the existing pervious soil layers beneath the foundation and down into the underlying faulty rock. The thickness of the permeable layers was about 5-6 metres and in order to reach these layers the bore holes had to be drilled from the crest and through the dam core to a depth of about 20 metres. It was also considered necessary to protect the bottom layers from possible erosion caused by leakage water flowing through cracks in the rock. For this purpose the grout curtain had to be extended to a depth of 10 metres below the rock surface after completion of the soil grouting.

After drilling the grout holes, the nuclear density probe was used for determining the initial porosity of the soil strata overlying the rock. From these

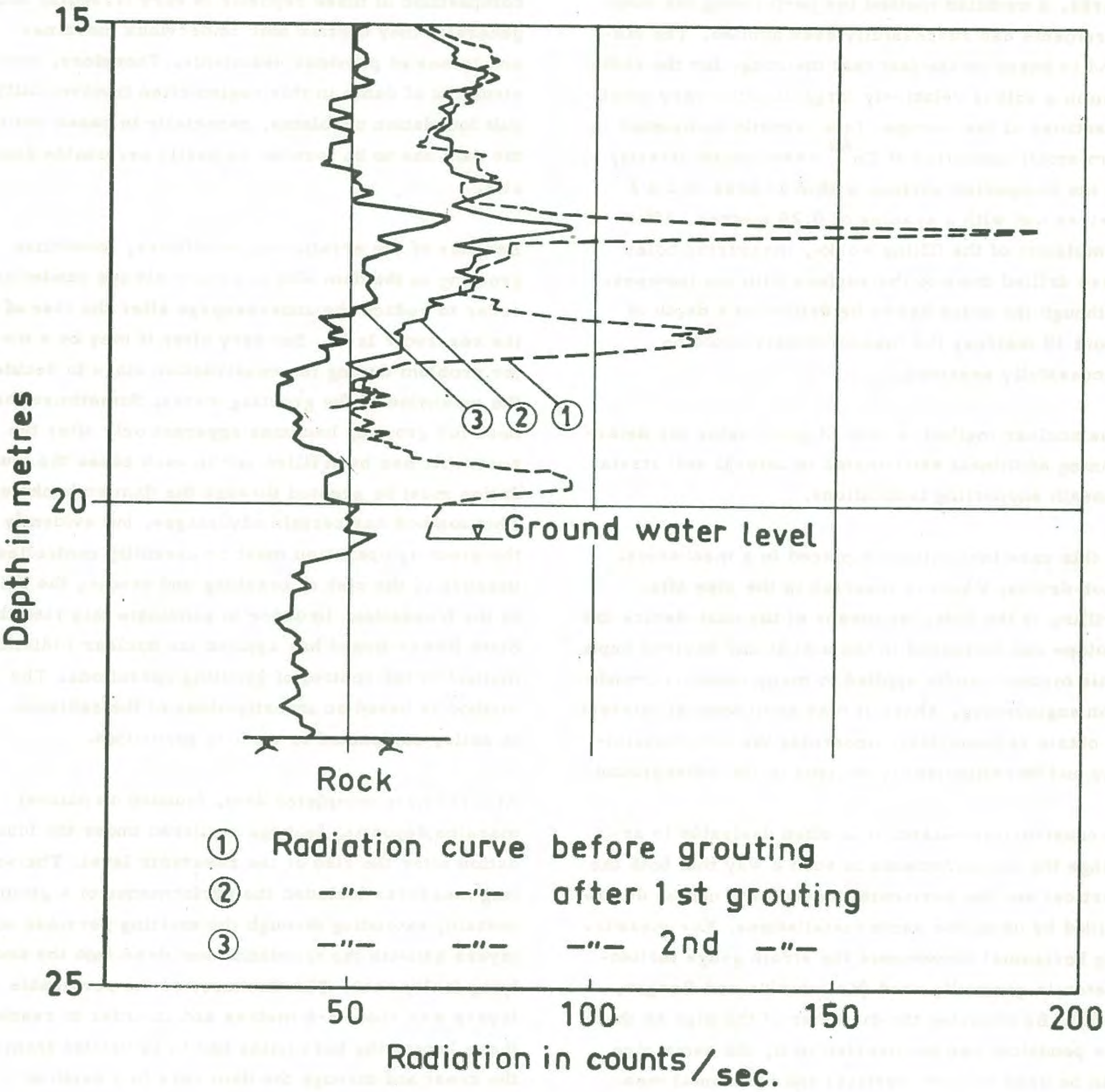


Fig. 2 Radiation curves, measured in foundation soils under a dam

investigations it was found that the soils, located immediately above the rock surface, were pervious and contained cavities, whereas the overlying soils in the contact zone against the dam foundation consisted of impervious moraines, As a result of this investigation it was decided to grout the holes in stages from the rock surface up to the bottom of the dam.

After completion of the soil grouting, secondary holes were drilled through the foundation to a depth of 5 metres in the rock. Before the rock grouting was begun, these holes were measured with the nuclear density probe for controlling the effect of the soil grouting. From the measurements, combined with the results of the conventional water pressure testing, valuable informations were obtained concerning the effect of grouting and also on many other factors which influence the grouting procedure in soils.

Fig. 2 shows a typical radiation diagram for a bore hole, drilled for grouting the foundation soils beneath the dam. In this case both the bottom layers and the rock were so pervious that the first grouting did not stop the flow of ground water into the bore hole. Therefore additional grouting was needed for sealing the pervious zones.

The same method was used for controlling the rock grouting and also in this case interesting results were obtained concerning the grouting procedure. Therefore it is believed that the nuclear method will be as valuable in the grouting technic as it is in many other fields of soil and rock mechanics.

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STABILITY OF NATURAL SLOPES AND EMBANKMENT FOUNDATIONS

STABILITE DES TALUS NATURELS ET DES FONDATIONS DE REMBLAIS

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Introduction

Several large landslides have occurred during the last 20 years in the densely populated Göta River Valley, located in the southwestern part of Sweden (Fig. 1). Among these the landslides at Surte (Jakobson, 1952) and at Göta (Odenstad, 1958) can be mentioned.

In the Göta River Valley flows the Göta River which is one of the largest rivers in Sweden. The Göta River is also an important waterway. Approximately 4 milj. tons of freight is annually transported on the river. In the river valley are located several hydro-electric plants. The river serves also as water supply for Gothenburg, the main seaport of Sweden with approximately 500,000 people. Two main highways follow the Göta River Valley. The highways are in many places located close to the river. Also one railroad line is located in the valley, and has in some cases been affected by landslides.

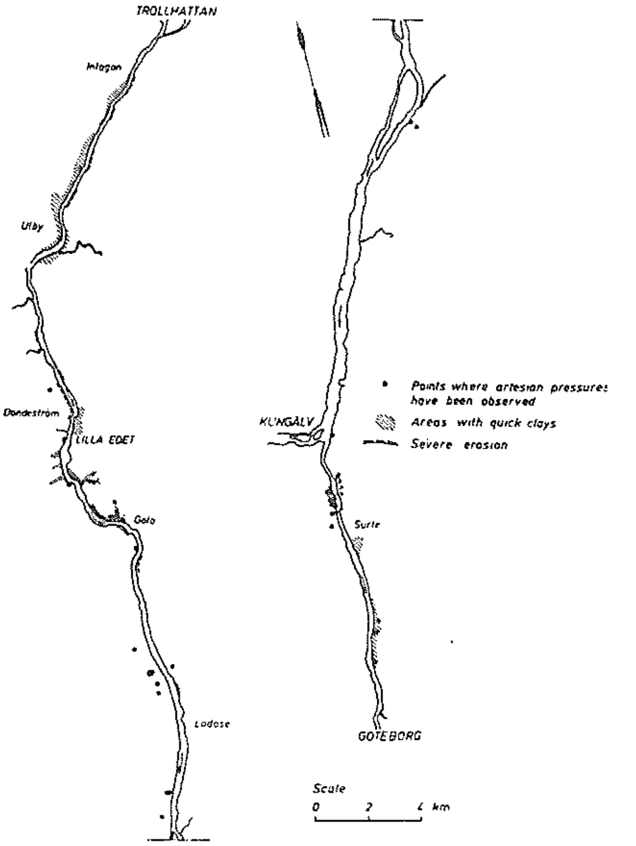


Fig. 1. The Göta River between Trollhättan and Gothenburg (Göteborg).

Landslides in the Göta River Valley can have very serious consequences. A landslide can, for example, disrupt the boat traffic in the river, interfere with the operation of the hydroelectric plants and pollute the river water. (Several large chemical plants are located at the river and oil storage tanks have been built close to the river. Two oil tanks collapsed e.g. during the slide at Göta.)

Landslides in the Göta River Valley

Scars after old landslides can be seen almost along the whole river, especially in the northern part of the valley above Lilla Edet. In Fig. 2 is shown scars after landslides at Utby (Fig. 1).

About 20 slides have here taken place within a distance of about 2 km. Several slides have also occurred along the Slump River, a tributory to the Göta River.

Scars after a large old landslide can also be seen at Jordfallet (in English "Earth fall") located about 2 km upstream of Surte (Fig. 1). The scar has a length of about 300 m and reaches about 1000 m from the river.

One of the largest recorded landslides occurred in 1648 at Intagan, located approximately 4 km south-

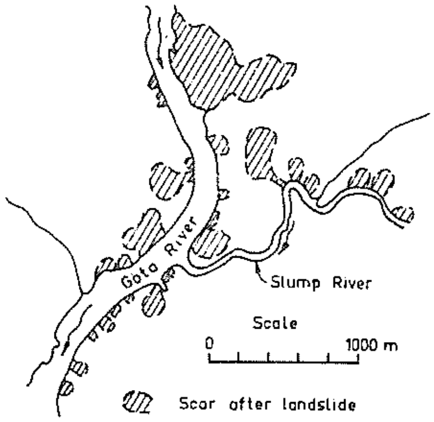


Fig. 2. Landslides at Utby.

west of Trollhättan (Fig. 1). About 54 acres were affected by the slide and at least 85 people lost their lives. Another relatively large landslide which probably was triggered by an earth quake took place in 1759 at Bondeström, when approximately 400,000 m^3 slid into the Göta River.

The landslide at Surte occurred in 1950. The direct cause of the slide is uncertain but the vibrations from the pile-driving for a small house have been dicussed as one possible factor as well as the vibrations from a train which passed the area just before the slide. The length of the slide area was about 600 m and the width about 400 m. The total volume was 4 million m^3. Within the slide area lived about 375 people in 31 houses. One person lost his life during the slide. A major highway and a railroad line, which crossed the slide area, were damaged. The total damage caused by the slide was estimated in 1952 to $ 2,000,000.

The landslide at Göta occurred in 1957. Several large buildings belonging to a paper mill were located within the slide area, which was about 1500 m long and 300 m wide. Three workers were killed of the 200 which were within the slide area. The total cost to restore the river channel after the slide was approximately $ 2,000,000.

Geologic History

The Göta River Valley was formed by erosion during several glaciations along prequaternary fissured zones in the underlaying bed rock of gneiss. The bottom of the valley, which is filled with loose sediments, is located up to 100 m below the present sea level. The maximum thickness of the sediments is about 100 m (at Gothenburg). The thickness of the sediments in general increases downstream. The area was covered during the last glaciation by ice with a total thickness of 2 000 to 3 000 m. During the last glaciation the valley floor was partly covered by a thin layer of heavily preconsolidated moraine (till) with a total thickness less than 1 to 2 m. The moraines are generally covered by glacifluvial and marine sediments. The average grain size of the sediments decreases generally towards the ground surface and downstream the river. The glacifluvial sediments (sand, silt and clay) and the moraines have in many places been reworked by waves.

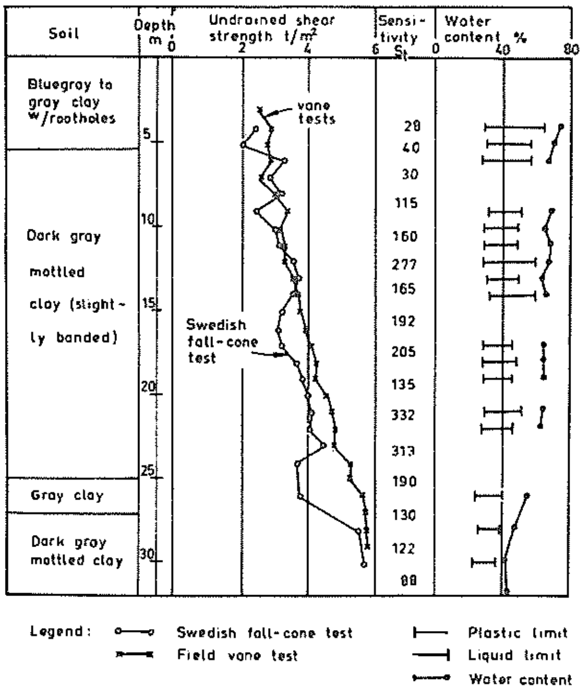


Fig. 3. Soil conditions at Utby.

The location of the shore line has changed considerably since the last glaciation, due to changes of the sea level and the general land uplift. At the end of the glaciation the sea level at Gothenburg was located 95 m above the present sea level and 128 m at Lake Vänern. The present rate of land uplift is 2 mm/year downstream at Gothenburg and 2.5 to 3 mm/year upstream at the Lake Vänern. The gradient of the Göta River is thus increasing by about 1 mm/year. A typical soil profile of the clay layer at Utby (Fig. 1) is shown in Fig. 3. (The clay is underlaid by sand with a thickness of more than 10 m.)

The lower parts of the marine clay sediments, which generally are stratified, are of glacial age. The clay content varies generally between 50 and 60 %. Organic content of the glacial clay is low. Varved silt can frequently be found close to the bottom layer of sand.

The upper parts of the marine sediments consist of late- or postglacial clays. The particle size is smaller than that of the underlaying glacial clay. The clay content varies generally between 60 and 70 %. The dominating clay mineral is illite. The organic content ("gyttja" and organic sulphides) is relatively low but increases downstream the river. The glacial as well as the late- and the postglacial clays are located just above Casagrande's "A"-line.

The post-glacial clays have a stiff upper crust with a total thickness of 1 to 1.5 m. The thickness increases upstream the river. The dry crust is disected by vertical desiccation cracks with a thickness varying between 1 and 6 mm.

Alluvial sediments can frequently be found close to the river. The silt and organic contents of these sediments are generally high. Their maximum thickness is about 7 m.

High artesian pressures have been measured in many places in the sand and silt layers as shown in Fig. 1. The water pressures are related to the amount of rain fall in the area. Seasonal variations of the ground water table of 1 to 2 m are not unusual. (Pore overpressures of 7 m with respect to the present ground surface has been observed.) The pore overpressures are generally highest in the bottom layers and decrease towards the ground surface.

The glacial and post-glacial clays are generally normally consolidated except for the stiff dry crust at the ground surface. The shear strength increases generally linearly with depth. Just below the dry crust the shear strength varies between 0.5 and 1.0 metric tons/m^2 in the downstream part of the valley and between 3 and 4 metric tons/m^2 in the upstream part of the valley.

The following relationship has been found for Swedish clays between undrained shear strength c_u, liquid limit w_L or fineness number w_F (Karlsson. 1961) and effective overburden pressure σ'_v (Hansbo, 1957).

$$c_u = 0.45\, w_L\, \sigma'_v$$

Thus the shear strength increases as a rule linearly with increasing liquid limit (or fineness number) and with increasing effective overburden pressure. The shear strength of the normally consolidated clays in the Göta River Valley is in some places higher than that predicted by the relationship proposed by Skempton (1954) or by Hansbo (1957) when the sensitivity of the clay is high (Karlsson & Viberg, 1967).

The angle of internal friction as determined by drained direct shear tests is approximately 22 à 23°. A higher angle of internal friction is generally obtained with drained triaxial tests than with drained direct shear tests (Osterman, 1960; Brink, 1967). The true angle of internal friction (Gibson, 1953) varies generally between 18° and 20° (Karlsson & Pusch, 1967).

The sensitivity of the clay in the Göta River Valley is frequently high. The sensitivity of the Swedish clays is normally about 10 to 20 but the sensitivity ratios as high as 500-700 has been measured by the fall-cone test (Karlsson, 1963) north of Utby. Often a relatively large amount of work is required to remould the clays from the Göta River Valley. Clays from the Göta River Valley are therefore not as sensitive to vibrations or other mechanical disturbances as clays which can be remoulded easily.

Söderblom (1969) has suggested to distinguish between rapid quick clays and slow quick clays (a clay is classified as rapid quick when a relatively small amount of work is required to remould the clay, while the term slow quick is used when the amount of work is relatively large). Clays with a sensitivity ratio larger than 50 are in Sweden defined as quick clays if the fall-cone test is used to determine the undisturbed and the remoulded shear strength of the soil.

The high sensitivity of the clays in the Göta River Valley can partly be explained by a reduction of the salt content due to leaching and diffusion. However, infiltration of organic substances (e.g. humus) have probably also contributed to the high sensitivity of the clays. The salt content of quick clays is always low but the salt content of clays with low sensitivity may vary between wide limits. Söderblom (1969) has identified a large number of organic substances in pore water with dispersing properties.

In Fig. 4 is shown the relationship between conductivity ($\varkappa$) and the sensitivity (S_t) of a clay from a bore hole located close to Utby (Fig. 1).

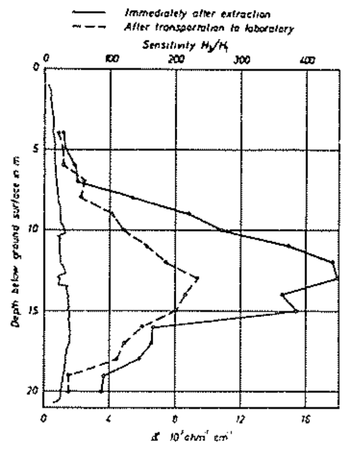


Fig. 4. Relationship between conductivity and sensitivity.

The conductivity, which is directly proportional to the salt content, was determined with a salt sounding tool (Söderblom, 1957). The shear strength of the undisturbed and of the remoulded soil was determined by the Swedish fall-cone test in the field immediately after the sampling and in the laboratory approximately two weeks after the sampling. It can be seen that no relationship exists in this case between conductivity (salt content) and sensitivity when the salt content is low. In fact, the highest sensitivities were found at the levels where the salt content was the highest.

The sensitivity is, however, affected by the time after sampling. The sensitivity determined two weeks after the sampling was approximately half the sensitivity immediately after the sampling. The reduction of the sensitivity was caused mainly by a reduction of the undisturbed shear strength of the clay.

In Fig. 5 is shown a microphotograph of a quick clay from Lilla Edet. The shear strength of the clay is about 2 t/m^2 and the sensitivity about 150. The thickness of the section is 500 Å. Investigations have shown that the microstructure of quick clays is characterized by an extremely open three dimensional network of particles and particle groups (Karlsson & Pusch, 1967).

Soil Exploration

The stability conditions in the Göta River Valley has been investigated extensively. The sounding machine type SGI (Fig. 6) and the Swedish weight sounding device have been used to determine the thickness of the individual soil layers and the depth to the firm bottom layers.

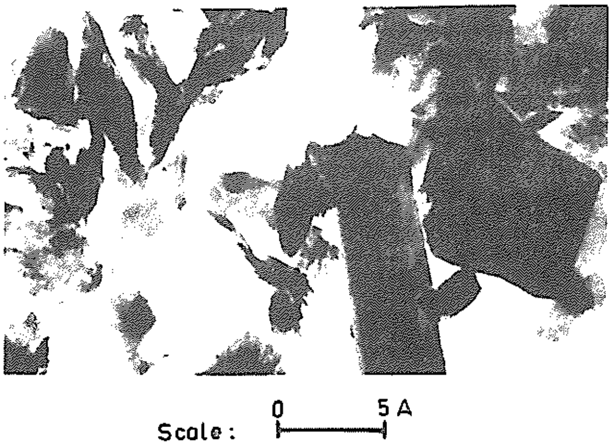


Fig. 5. Microphotograph of a quick clay from Lilla Edet (R. Pusch).

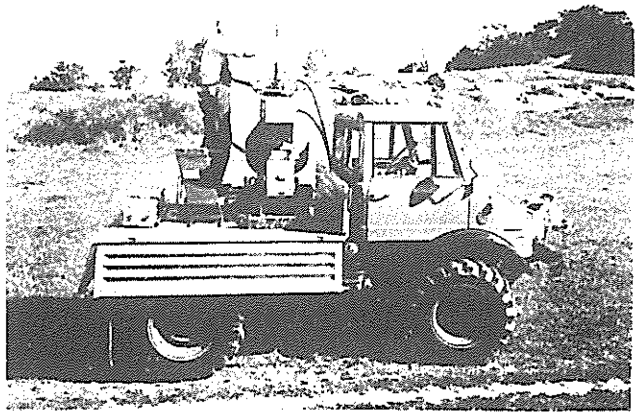


Fig. 6. Sounding machine type SGI.

The sounding machine type SGI (Kallstenius, 1961) records automatically the penetration resistance of a 10 cm^2 cone as a function of depth as the cone is pushed down into the soil. The skin friction resistance is automatically subtracted from the total resistance and only the point resistance is recorded by the machine. This type of penetrometer is much more rapid than, for instance, the weight sounding test, and can be used to detect e.g. sand and silt seams as shown in Fig. 7.

The undisturbed shear strength of the clay has been determined in situ with the so-called iskymeter (Kallstenius, 1961) and with vane tests and in the laboratory by the Swedish fall-cone (Hansbo, 1957) or by unconfined compression tests. The iskymeter records the pull-out resistance of a foldable anchor plate which is pulled out of the ground by a wire. The anchor plate is driven into the soil folded and unfolds when the plate is pulled. With the iskymeter sand or silt seams can be detected as shown in Fig. 8.

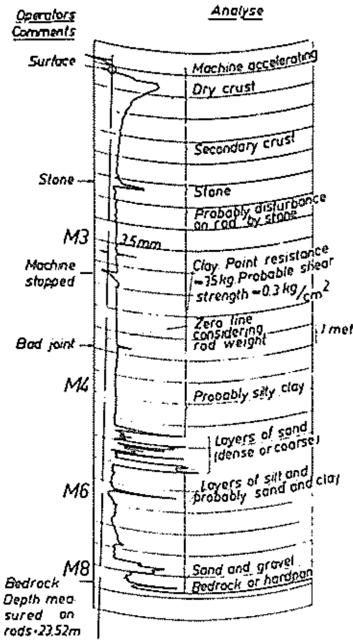


Fig. 7. Penetrometer record.

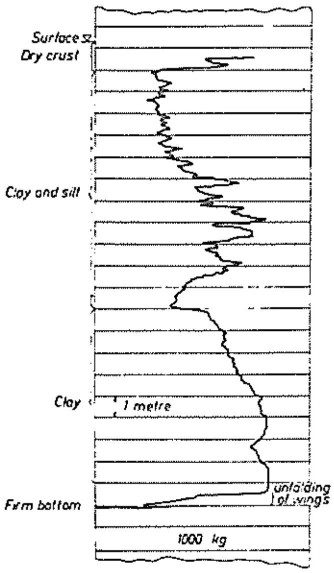


Fig. 8. Iskymeter record.

The shear strength determined by the iskymeter agrees relatively close with the results from vane tests in clays with normal sensitivity (Osterman, 1960). A good agreement has also been found between fall-cone and field vane tests except when the sensitivity of the soil is high.

The vane type SGI is provided with shield or with a shroud which protects the vane blades during driving. The shield also prevents clay from adhering to the blades when the vane is driven through the stiff dry crust. The test results indicate that clay which adheres to the vane blades can appreciably lower the measured shear strength especially when the sensitivity of the clay is relatively high.

Soil samples have been taken with sampler SGI type IV (Kallstenius, 1958) and by the Swedish standard piston sampler (Swedish Committee on Piston Sampling, 1961). The area ratio of the Swedish standard piston sampler is low. Samples have been taken every or every second meter in depth.

The Swedish foil sampler has been used in a few cases to check the layer sequence and to locate slip surface in slides. In Fig. 9 is shown two cores from the slide at Göta. The location of the slip surfaces can clearly be seen on the photograph.

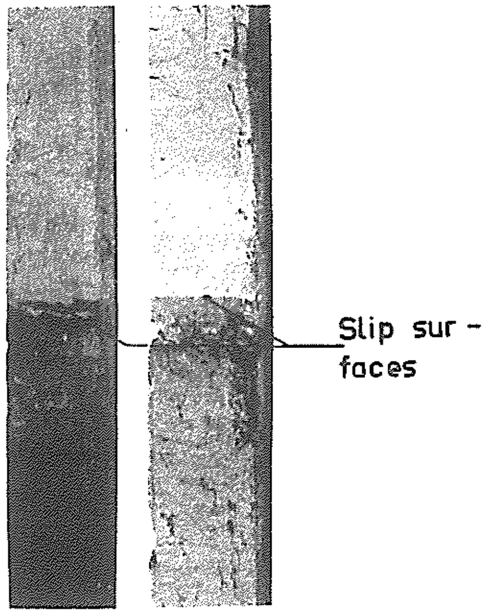


Fig. 9. Location of slip surfaces in cores.

Pore pressures have been measured hydraulically by pore pressure gauges of type SGI (Kallstenius & Wallgren, 1956). The coupling device of the gauge makes it possible to check the functioning and the accuracy of the gauges during long time measurements. These gauges are therefore especially suited for control of pore pressures. The gauges along the Göta River are as a rule read every second month.

Lateral displacements (creep) have been measured with an inclinometer of type SGI (Kallstenius & Bergau, 1961) at locations where the slope stability has been especially low. With this device it is often possible to measure lateral displacements with an accuracy of about 1 to 2 mm for a length of 10 m.

The salt content of the clays has been determined at a large number of places by measuring in situ the electrical resistance of the soil with a salt sounding tool as mentioned above (Söderblom, 1957, 1958).

The electrical resistance is measured between two isolated steel rings which are placed at the tip of a sounding rod. The tool is driven into the ground with standard equipment. The salt content, which generally is low close to the ground surface, increases with depth and reaches often a maximum value at the middle of the profile. The salt content at the bottom layers is generally low due to leaching. It has been possible in many cases to locate slip surfaces with this device. In Fig. 10 is shown the record from a salt sounding at Göta which was carried out 14 days after the slide (Söderblom, 1969).

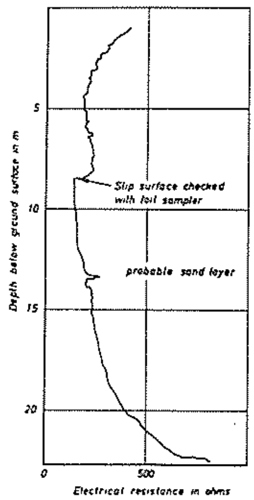


Fig. 10. Location of slip surface with the salt sounding tool.

A sudden jump in the resistivity curve shows the location of the slip surface. The actual location was later checked with cores obtained with the Swedish foil sampler.

Attempts have also been made to locate slip surfaces in quick clays with field vane tests. This method has not been successful since the thickness of the disturbed zone, where the shear strength in the soil is reduced by remoulding, has been small (less than a few centimeters).

Slide Mechanisms

The large slides in the Göta River Valley are generally retrogressive rotational or retrogressive translational. They have as a rule been triggered by a relatively small initial local slide. A number of parallel ridges can often be seen within the area after a slide. The landslide at Göta started as a small local slip close to the river. In the morning of the slide a crack which was 130 m long was observed approximately 20 to 30 m from the river. The crack widened slowly and the final slip occurred 3 hours after the crack first was detected. The slide spread upstream the river and in the direction from the river (Odenstad, 1958). The slide at Surte initiated, however, at some distance from the river and the slide then spread gradually towards the river and backwards to the side of the valley. The duration of the slide was at Surte about 3 minutes and the maximum speed of the slide was less than 0.9 m/sec.

A number of local slide can be seen in Fig. 11.

Fig. 11. Local slides along the Göta River.

The small initial slide is generally caused by erosion of the river bank and by changes of the ground water level, by vibrations from traffic, pile driving or by earth quakes. The small local slides are mainly confined to the part of the river located above Lödöse (Fig. 1) where the height of the river banks exceeds several meters.

The erosion processes have accelerated during the last 50 years partly by the construction of dams and hydroelectric plants along the river and partly by the heavy ship traffic on the river. The erosion of the river banks seems to increase rapidly with the size and speed of the ships. Some bottom erosion has taken place downstream of Lilla Edet at the places where the river bottom consists of cohesionless soils. Of course, a certain erosion has also been caused by the general land uplift (2 mm/year at Gothenburg). This erosion is relatively small since the land uplift is small. Over a very long time period this bottom erosion can, however, be appreciable and contribute to the general lowering of the stability of the slopes.

Method of Analysis

It is not possible to predict accurately with presently available methods if a small local slide will develop into av large catastrophical slide. The stability of a number of small local slides have been analysed by the total stress method (ϕ = 0-analysis) and by the effective stress method ($c'\phi'$-analysis) (Bishop, 1955). The total stress analysis which is based on results from field vane tests gave nominal safety factors which are close to one with respect to available and mobilized shear strength of the soil (Götaälvkommittén, 1962). Safety factors of the same magnitude were obtained from an effective stress analysis if the results from drained direct shear tests and triaxial tests are used in the analysis. The cohesion c', determined by drained direct shear test, has not been reduced in the calculation. Both circular and other shapes of the failure surfaces have been investigated.

Also sections where slides have not occurred have been analysed by the total and the effective stress methods. The results from these calculations are shown in Table 1.

Table 1. Safety factors calculated by total and effective stress analysis

Location	Total stress analysis	Effective stress analysis
Intagan - Torp		
23.45	1.18	1.30
23.72	1.02	1.03
23.82	1.07	1.03
26.88 - 27.52	~ 1.1	~ 1.1
Ströms Lock		
37.01	1.0	1.0
37.11	1.1	1.0
37.33	1.0	1.0
Lilla Edet		
34.5 - 35.8	1.4 - 1.5	1.3 - 1.5
Agnesberg - Ekeberg	~ 1.5	1.3 - 1.5

Also in this case a good agreement was obtained between the two methods.

The stability of the normally consolidated clay in the Göta River Valley is generally evaluated by the total stress method (ϕ = 0-analysis) mainly due to the simplicity of this method. A total stress analysis reflects the stability of a slope with respect to sudden changes of the applied load and to gradual changes of the undrained shear strength of the soil by e.g. leaching. It is believed that a total stress analysis to a certain extent can be used to investigate if a small slide will cause a large secondary slide. (The initial slide causes a sudden change of the stress conditions in the remaining soil without a change of the water content.) Erosion of the river banks often occurs rapidly due to large variations of the water level and the heavy boat traffic in the Göta River. The change of the stress conditions by the erosion will generally cause an increase of the pore pressures in the slope due to the high sensitivity of the clay and the high value of the pore pressure coefficient A. Dissipation of excess pore pressures and changes of the water content of the clay take place very slowly due to the low permeability of the clay (10^{-8} to 10^{-10} cm/sec). It is believed that changes of the stability conditions by such rapid changes of the stress conditions is better reflected by a total stress analysis than by an effective stress analysis. In this case a total stress analysis will give a lower safety factor than an effective stress analysis. The limitation of a total stress analysis is, however, the evaluation of the average undrained shear strength of the soil. It is known that anisotropy, and the direction of the stress increase (Bjerrum & Kenney, 1967), and thus the test method will affect the measured shear strengths.

The undrained shear strength determined by the field vane or by the Swedish fall-cone test are used in the calculations as mentioned above. The shear strength determined by vane tests agrees as a rule closely with the cone values because the cone test has been calibrated against the vane test. If the sensitivity of the soil is high and the sampling depth is large, the vane shear strength is, however, frequently higher than the cone strength because of the disturbance of the soil samples.

The actual shear strength is probably higher than the vane or cone values. For normally consolidated clays the shear strength along horizontal planes is often higher than along vertical planes. This difference in shear strength is probably compensated by the strength decrease caused by time.

The shear strength used in the calculations is reduced when the liquid limit or fineness number of the clay is larger than 80 or the soil is organic. A reduction coefficient of 0.9 is often used for slightly organic clays, 0.8 for organic clays and silts, 0.7 for clayey or silty "gyttja" and 0.6 for peat and "gyttja". ("Gyttja" is an organic soil of postglacial or recent age.) Analysis of old slides which have occurred in organic soils with a high liquid limit has indicated that a reduction is required.

In Sweden normally a safety factor of at least 1.4 to 1.5 is required with respect to the undrained reduced shearing strength of the soil in areas where the stability conditions can be changed by the construction of e.g. buildings and roads close to the river or by erosion. In areas in the Göta River Valley where the stability conditions are supposed not to change in the future and where structures are not located close to the river a nominal safety factor of 1.3 is generally considered satisfactory. In some cases a safety factor as low as 1.2 has been allowed. If safety factors of 1.3 to 1.2 are found it is important to protect the slopes from erosion and to prevent buildings to be constructed near the river or to deepen the river channel by e.g. dredging. Where the factor is lower than the above mentioned the actual stability has been increased by about ten per cent through unloading of the river bank by excavation.

An effective stress analysis ($c'\phi'$-analysis) is especially valuable to check the stability conditions in the Göta River Valley when gradual changes of the loading conditions or of the pore pressures are expected. It is often difficult to predict the magnitude of these changes. Furthermore, there are some difficulties in the evaluation of the coefficients ϕ' and c' which are used in an effective stress analysis. These coefficients are affected by volume changes during the drained triaxial and the direct shear tests, by rotation of the principal stress directions and to a minor extent by the intermediate principal stress. There are also some questions about the value of the parameter c' to be used in the analysis. The parameter c' is not reduced at present since a reduction leads to unreasonable results. (If a reduced value is used in the calculations, then large parts of the Göta River Valley are not stable according to the calculations.)

Slope Protection

The slopes along large parts of the Göta River do not satisfy the safety requirements mentioned above. Therefore extensively remedial works have been or are being carried out along the river. By protecting the slopes from local slides it is believed that the

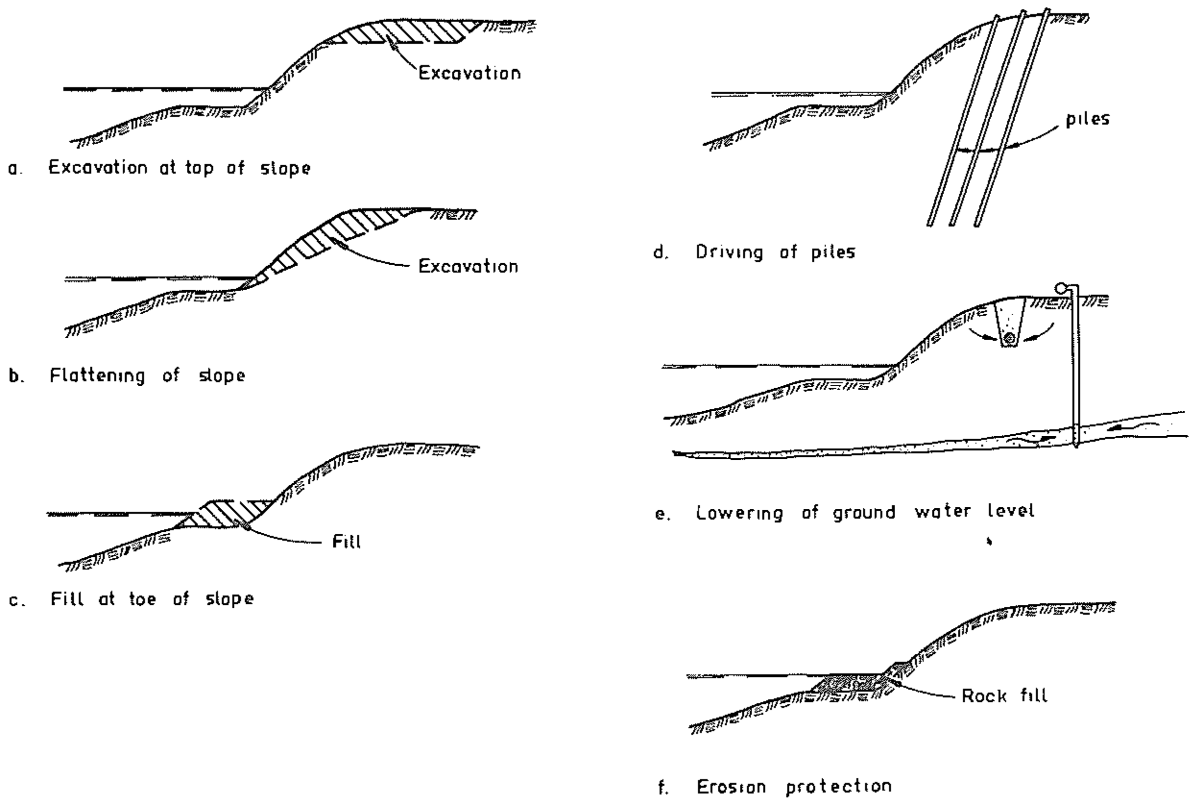


Fig. 12. Stabilization of slopes.

slopes will also be safe against large landslides.

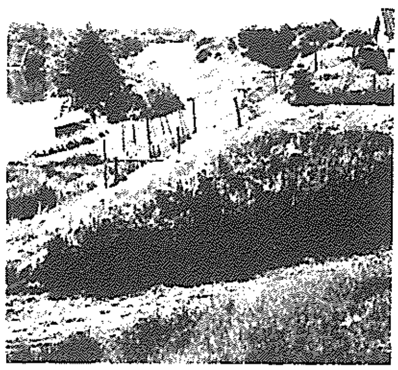
The remedial works can be separated into three major groups. To the first group belong all methods which tend to decrease the nominal average shear stress along potential slip or failure surfaces. The average shear stress can be decreased e.g. by excavating the soil at top of a slope, by flattening the slope, by placement of a fill at the toe of a slope or by raising the water level in the river as illustrated in Fig. 12 a through c. To this group belongs also strengthening of the slope with piles (Fig. 12 d).

The second group includes methods which tend to increase the shear strength of the soil along potential slip surfaces. The shear strength of the soil can be increased e.g. by lowering the pore pressures by drainage or by pumping (Fig. 12 e). To the third group belong methods which prevent a decrease of the slope stability by e.g. erosion. Erosion can be prevented by a reversed filter at the toe of the slope as illustrated in Fig. 12 f.

The method chosen at a particular location is dependent of the local conditions such as the height and the slope of the river banks, the presence of permeable layers in the soil, the average shear strength of the soil and of the erosion.

It is generally much more effective to remove part of the soil at the top of the slope as shown in Fig. 13, than to flatten the slope or to place a fill at the toe of the slope.

A fill at the toe of the slope is often partly located below the water level where only the boyant weight of the fill is effective.



13. Increase of slope stability by excavation at top of slope.

Fig. 14. Slide caused by the driving of piles.

Piles are seldom used to increase the slope stability except when the space at the top of the slope is restricted by an adjacent road or by buildings. However, the driving of piles may temporarily decrease the slope stability by the remoulding of the soil or by the lateral displacement of the slope towards the river. A slide caused by pile driving is shown in Fig. 14. In this case the lateral displacements and the resulting shear deformations of the soil were sufficiently large, so that the peak strength of the soil was exceeded (Broms & Bennermark, 1968).

If the sensitivity of the soil is high, the reduction of the shear strength at large lateral displacements can be large. However, due to the relatively high clay content of the clay in the Göta River Valley the reduction of the shearing strength is generally moderate. Experience has shown that the lateral displacement and the strength reduction is decreased if the row of piles located close to the river are driven first.

The ground water pressure can generally only be lowered where silt or sand layers are present in the soil. In areas with artesian pressures the ground water pressure can be lowered with bleeder wells. In all other cases the ground water pressure can only be lowered by pumping. However, a lowering of the ground water pressure causes settlements and may cause wells in the area to go dry.

The erosion protection is often combined with a flattening of the slopes to prevent local failures as mentioned previously. An erosion protection consists generally of a 2 m thick layer of rock fill which is placed by dumping (Fig. 15).

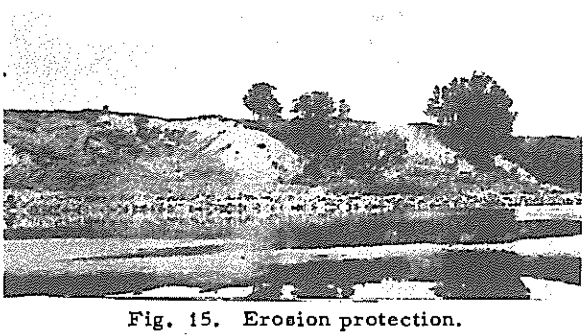


Fig. 15. Erosion protection.

The fill is not compacted. The maximum particle size is at least 15 cm. Only the sections where the erosion at present is very active is protected. However, a change of the local conditions (e.g. a change of the maximum water flow or of the ship traffic) may change the areas affected by erosion.

Slide warning systems have been installed by the Swedish State Railways (S. J.). The warning system consists of a thin electric wire with a tensile strength less than 110 lb, which is placed in the ground in a zig-zag pattern as illustrated in Fig. 16 at a depth of 1.0 m. The train traffic is stopped automatically when the cable breaks.

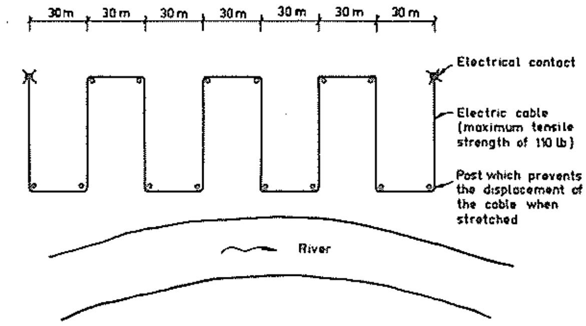


Fig. 16. Slide warning system type Swedish State Railways.

Twice each year the Göta River Valley is inspected by personal from the Swedish Geotechnical Institute. The erosion of the river banks is followed with aerial photographs. The erosion of the river bottom is checked by soundings. Also the construction of new buildings, storage tanks, the placement of fills etc. close to the river is controlled by the Institute.

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VIBRATORY COMPACTION OF COHESIONLESS SOILS

by
Bengt B. Broms and Lars Forssblad

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VIBRATORY COMPACTION OF COHESIONLESS SOILS

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Lars Forssblad

I. INTRODUCTION

Vibratory soil compaction was first studied and applied in Germany during the 1930's (Loos, 1936). A self-propelled caterpillar-type vibrating plate compactor with a total weight of about 25 tons was developed by Losenhausenwerk as early as 1933. This vibrator had a variable frequency between 10 and 25 cps and a maximum depth effect of 2.0 to 2.5 m (Müller & Ramspeck, 1935; Ramspeck, 1937). Losenhausenwerk also designed at about the same time a self-propelled 1.5 ton vibrating plate compactor.

The vibroflotation method where heavy internal vibrators in combination with jetting are used to compact cohesionless soils to large depths was also developed in Germany during the 1930's (Schneider, 1938).

Other types of vibratory soil compactors were developed during the 1940's such as tractor-drawn and self-propelled vibrating rollers, multiple plate compactors and vibrating tampers (rammers) as described e.g. by Garbotz (1958, 1959), Erlenbach (1959), Kronenberger (1960), Leussink (1960) and Forssblad (1965).

Vibratory soil compactors can be classified as indicated in Table 1. Vibrating tampers are light, hand-guided machines operating at a low frequency, about 10 cps, and with a large amplitude. The types of self-propelled vibrating plate compactors which are most commonly used today weigh between 100 and 500 kg.

Table 1. Different types of vibratory soil compactors

Surface vibrators			Internal vibrators		
Type of machine	Weight	Frequency	Type of machine	Diameter	Frequency
Vibrating tampers (rammers)			Concrete vibrators		
Hand-guided	50-150 kg (100-300 lb)	about 10 cps	Manually operated or tractor-mounted	5 - 15 cm (2 - 6 in)	100 - 200 cps
Vibrating plate compactors			Vibroflotation equipm.		
Self-propelled, hand-guided	50-3000 kg (100-6000 lb)	12 - 80 cps	Crane-mounted	23 - 38 cm (9 - 15 in)	about 30 cps
Multiple-type, mounted on tractors, etc.	200- 300 kg (400- 600 lb)	30 - 70 cps			
Crane-mounted[1)]	up to 20 ton	10 - 15 cps			
Vibrating rollers					
Self-propelled, hand-guided (one or two drums)	250-1500 kg (500-3000 lb)	40 - 80 cps			
Self-propelled, tandem-type	0.7- 10 ton	30 - 80 cps			
Self-propelled, driving rubber-tires	4 - 25 ton	20 - 40 cps			
Tractor-drawn	1.5- 15 ton	20 - 50 cps			

1) Used on a limited scale

Self-propelled vibrating rollers are of three main types - hand-guided, vibrating tandem rollers and vibrating rollers with two driving rubber-tires. Tractor-drawn vibrating rollers with weights up to 15 tons and vibrating rollers with driving rubber-tires with total weights up to 25 tons are nowadays used.

The use of vibratory soil compaction has increased considerably during the last 5 to 10 years. Vibratory soil compaction was initially used only for cohesionless soils (Lange, 1940; Leussink, 1951). Nowadays vibratory compactors are also used to compact cohesive soils and asphalt surfaces. The main applications are given in Table 2.

The knowledge of the factors affecting vibratory compaction of soil is still incomplete in spite of the extensive investigations which have been carried out all over the world since the 1930's. The compaction of cohesionless soils is affected e.g. by average grain size, grain size distribution, shape of particles, water content, permeability, compressibility, shear strength, etc. Also the properties of the vibratory compactor such as frequency, nominal amplitude, centrifugal force and total weight affect the compaction. The descriptions of the soil and of the compactor in test reports and technical articles are usually very schematic.

The many factors which affect the vibratory compaction make it difficult to draw general conclusions. It is therefore important to study carefully the actual test and soil conditions before test results are applied to other conditions than those under which the tests were carried out.

This report deals mainly with vibratory compaction of cohesionless soils, but some data and results from investigations on rock fills and cohesive soils are given for comparisons.

Table 2. Applications of vibratory soil compaction

Type of machine	Applications
Vibrating tampers (rammers)	Street repair. Fills behind bridge abutments, retaining and basement walls, etc. Trench fills.
Vibrating plate compactors	
Self-propelled, hand-guided	Base and subbase compaction for streets, side-walks, etc. Street repair. Fills behind bridge abutments, retaining and basement walls, etc. Fills below floors. Trench fills.
Multiple-type	Base and subbase compaction for highways.
Vibrating rollers	
Self-propelled, hand-guided	Base, subbase and asphalt compaction for streets, side-walks, parking areas, garage driveways, etc. Fills behind bridge abutments and retaining walls. Fills below floors. Trench fills.
Self-propelled, tandem type	Base, subbase and asphalt compaction for highways, streets, side-walks, parking areas, garage driveways, etc. Fills below floors.
Self-propelled, driving rubber-tires	Base, subbase and embankment compaction for highways, streets, parking areas, air fields, etc. Rock fill dams. Fills (soil or rock) used as foundations for residential and industrial buildings.
Tractor-drawn	Base, subbase and embankment compaction on highways, streets, parking areas, air fields, etc. Earth- and rock fill dams. Fills (soil or rock) used as foundations for residential and industrial buildings. Deep compaction of natural deposits of sand.

II. ELEMENTS OF SOIL COMPACTION

The purpose of compaction is to improve the strength and deformation properties of soils. The shear strength of cohesionless soils increase while the compressibility and the permeability decrease with increasing density. For example, the angle of internal friction of sand or gravel increases with up to 12° when the relative density increases from loose to dense.

The compressibility of cohesionless soils is also affected by small changes of the density. Compaction of a soil will often increase the equivalent modulus of elasticity as determined by plate load tests 5 to 10 times.

The permeability is also affected appreciably by the relative density. When the relative density increases from loose to dense the permeability of a uniform sand or gravel decreases by about 50 to 60%. The change of the permeability increases with increasing coefficient of uniformity and with decreasing grain size (Taylor, 1948).

The compaction of a soil is in most cases expressed by the relative compaction or by the relative density. The relative compaction or the degree of compaction R_D (%) is defined by the equation:

$$R_D = \frac{\gamma_d}{\gamma_{d,max}} \times 100 \qquad (1)$$

where γ_d is the actual dry density (dry unit weight) and $\gamma_{d,max}$ is the maximum dry density determined by the Proctor compaction tests (standard AASHO, modified AASHO, etc.) or by vibratory compaction methods as illustrated in Fig. 1.

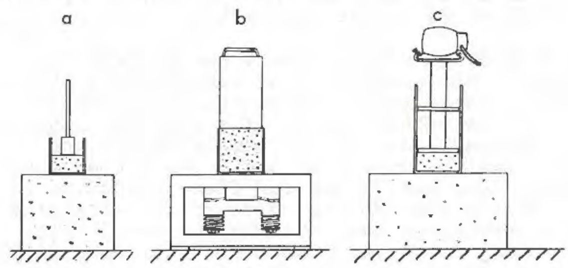


Fig. 1. Laboratory Compaction Methods a) Proctor Compaction Test; b) Vibratory Compaction Test with Vibrating Table and Surcharge Load; c) Vibratory Compaction Test with Vibrating Tamper or Hammer

The compaction of cohesionless soils is often expressed by the relative density. The relative density I_D (%) of a soil with the void ratio e is defined by:

$$I_D = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100 \qquad (2)$$

In this equation e_{max} and e_{min} are the maximum and the minimum void ratio of the soil, respectively. The relative density is equal to zero when the soil is loose ($e = e_{max}$) and 100% when the soil is dense ($e = e_{min}$). For a soil with the dry density γ_d Eq. (2) can be rewritten as:

$$I_D = \frac{\gamma_{d,max} \, (\gamma_d - \gamma_{d,min})}{\gamma_d \, (\gamma_{d,max} - \gamma_{d,min})} \times 100 \qquad (3)$$

where $\gamma_{d,max}$ and $\gamma_{d,min}$ are the maximum and minimum dry density of the soil, respectively.

A vibratory compaction method for cohesionless, free-draining soils has been developed by the Bureau of Reclamation (1963). This method has been approved as ASTM Standard D 2049. The maximum dry density is determined by vibrating the soil either saturated or completely dry for 8 minutes in a test mould under a surcharge load (Fig. 1 b). The volume of the mould is 0.1 cu. ft. alternatively 0.5 cu. ft.

The maximum void ratio e_{max} and the minimum density $\gamma_{d,min}$ are usually determined by pouring dry soil through a funnel into a Proctor mould. The funnel is generally held 25 mm above the soil surface. Soils with particles larger than 10 mm are generally placed with a scoop (Bureau of Reclamation, 1963).

The maximum density can also be determined by laboratory compaction tests using a vibrating hammer or a vibrating tamper working on the surface of the material (Fig. 1 c). The method using a vibrating hammer has been developed by the Road Research Laboratory in England. (Parsons, 1964; Odubanjo, 1968).

The vibrating tamper method has been developed by Vibro-Verken (Forssblad, 1965, 1967). According to this method the soil is compacted in two layers in a mould with 15 cm diameter. Each layer is vibrated for 2 min. The resulting maximum dry density is for saturated cohesionless soils about the same as that determined by the modified AASHO compaction test.

The compaction water content has a different effect on the compaction of cohesionless, free-draining soils than on fine-drained, cohesive soils. The water content of a free-draining soil decreases during the compaction if the initial water content is high, while the water content does not change for fine-grained soils. As a result the Proctor curve of free-draining soils terminates near the maximum value (Fig. 2). If a sandy or gravelly soil contains some silt or clay, it is not free-draining, and excess water does not drain out during the compaction. The resulting compaction curve is regular as shown in Fig. 2 by Curve 2. Cohesionless free-draining soils very often have comparatively flat compaction curves as illustrated in Fig. 2 by Curves 1, 3 and 4. In some cases two maxima are obtained probably because of capillary forces in the partially saturated soil.

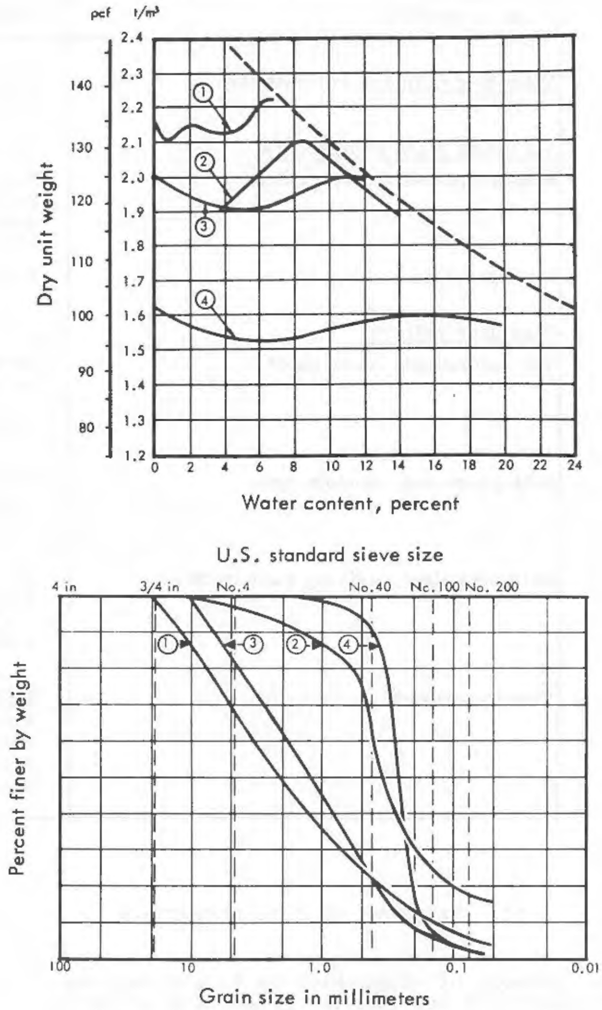


Fig. 2. Compaction Curves Obtained with the Modified AASHO Method and Grain Size Distributions for Different Types of Sandy Soils

Due to the comparatively flat compaction curves for free-draining soils and the fact that excess water can drain out during the compaction, the water content of the soil is not as important for cohesionless, free-draining soils as for fine-grained soils. The best compaction of sand and gravel is usually obtained when the soil is saturated or completely dry, but a comparatively good compaction can frequently be obtained also when the soil is partially saturated.

Investigations by Bureau of Reclamation (1963) and others as well as practical experience indicate that a soil usually can be characterized as free-draining if the content of fines (soil particles with a diameter smaller than 0.06 or 0.074 mm) is less than 5 to 10%. In special cases soils can act as free-draining when the content of fines is as high as 15 to 20%.

However, well-graded, sandy and gravelly soils (silty sand, clayey gravel, etc.) with more than 5 to 10% of fines are as a rule not free-draining. These soils have certain cohesive properties, and the control of the compaction water content is essential (Fig. 2).

The relevance of the Proctor compaction tests to vibratory compaction of cohesionless soils has been questioned by many investigators (Lane, 1948; Felt, 1958; Morris & Cochrane, 1965). It has been suggested that vibratory soil compaction should be correlated with laboratory compaction tests based on vibration.

Impact compaction tests such as the Proctor compaction tests are generally time consuming in comparison with vibratory compaction tests. In addition impact tests are to a certain extent affected by the manual performance.

A further advantage with vibratory compaction tests is that such tests can be adapted to test moulds with large diameters and can therefore be carried out on samples containing stones.

III. TECHNICAL DATA OF VIBRATORY COMPACTORS

There are no accepted rules regarding the technical data which should be specified or given for vibratory compactors in specifications, technical reports, etc. Sometimes such more or less undefined terms as "dynamic force", "dynamic load", "compaction effort in tons", "impulse in tons per second", "equivalent static weight" are used.

The vibrations of the drum of a vibrating roller or of the bottomplate of a vibrating plate compactor are usually generated by one or several rotating eccentrics. The frequency, amplitude, velocity and acceleration of a sinusoidal oscillation are defined in Fig. 3. The frequency expressed in cycles per minute (cpm) or cycles per second (cps or Hz) is determined by the rotational velocity of the eccentric(s). The amplitude of the drum or the bottomplate is a function of the eccentric moment of the eccentric as shown in Fig. 4 and of the mass of the vibrating system.

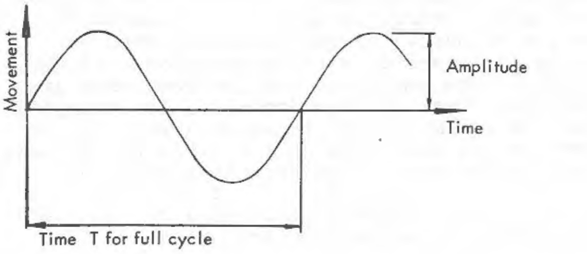
The so-called nominal amplitude can be calculated from the following equation:

$$\text{Nominal amplitude} = \frac{\text{Eccentric moment}}{\text{Mass of drum or bottomplate}} \quad (4)$$

The actual amplitude can for certain frequency ranges be up to 50% to 100% larger than the nominal amplitude due to resonance.

The rotating eccentric initiates a centrifugal force which is a function of the frequency and the eccentric moment as shown in Fig. 4. (Due to the oscillations of the drum or bottomplate there will be a small difference between the real centrifugal force and that calculated from Fig. 4. The difference is usually less than 1%).

The centrifugal force acts inside the drum or bottomplate. Its magnitude is not equal to the dynamic force transmitted to the underlying soil as is sometimes assumed.



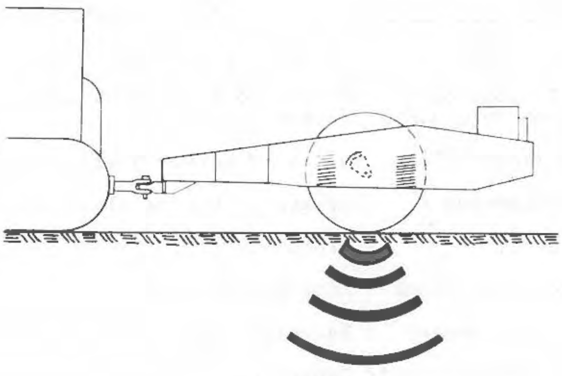
Frequency (number of cycles per sec. ($^1/T$)) = f cps or Hz

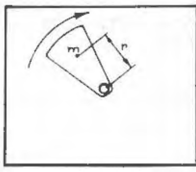
Amplitude (maximum deviation from position at rest) = s cm (in)

Maximum velocity = $v_{max} = 2\pi \cdot f \cdot s$ cm/s (in/sec)

Maximum acceleration = $a_{max} = 4\pi^2 \cdot f^2 \cdot s$ cm/s²(in/sec²)

Fig. 3. Definition of Frequency, Amplitude, Velocity and Acceleration for a Sinusoidal Oscillation





Weight of eccentric = m kg (lb)

Eccentricity = r cm (in)

Eccentric moment = m · r kgcm (lb in)

Centrifugal force = $m \cdot r \cdot 4\pi^2 \cdot f^2 \cdot \frac{1}{g}$ kg (lb)

g = acceleration due to gravity = 981 cm/s² = 386 in/sec².

Fig. 4. Definition of Eccentric Moment and Centrifugal Force

Through the oscillations defined by their frequency and amplitude the vibrating drum or bottomplate affects the surface of the ground with a rapid succession of dynamic loads or impacts. Each load cycle generates a stress wave as illustrated in Fig. 4. It is important to make a clear distinction between the vibrations of the compactor and the characteristics of the stress waves in the soil. The characteristics of the stress waves are to a large extent dependent on the properties of the soil.

The static weight of vibratory compactors has a large influence on the compaction effect since the kinetic energy as well as the momentum (mass x velocity) of the vibrating drum or bottom plate are directly proportional to the weight if the amplitude and the frequency are constant. Test results indicate, for example, that the maximum depth to which a soil can be compacted is depending on the total static weight of the compactor. However, the compaction close to the surface is for vibrating rollers also to a large extent influenced by the static weight per linear inch of drum width.

The ratio of drum and frame weight is also of importance with respect to the compaction effect. However, this weight ratio is given in specifications, technical reports, etc. for vibrating rollers and vibrating plate compactors.

It is proposed that at least the technical data in Table 3 should be given in specifications, technical reports, etc. for vibrating rollers and vibrating plate compactors.

TABLE 3. -- TECHNICAL DATA FOR VIBRATING ROLLERS AND VIBRATING PLATE COMPACTORS

Vibrating rollers	Vibrating plate compactors
Total weight	Total weight
Static load per inch of drum width	Static load per unit area of bottom plate
Drum width	Width of bottom plate
Drum diameter	Length of bottom plate
Frequency	Frequency
Nominal amplitude	Nominal amplitude
Eccentric moment	Eccentric moment
Centrifugal force[1)]	Centrifugal force[1)]
Working speed	Working speed

1) Can be misleading if compactors with two different frequencies are compared.

IV. COMPACTION MECHANISMS

A large number of factors affect the vibratory compaction of cohesionless soils. According to Lorenz (1955, 1960) the most important of these factors are the resonant frequency of the compactor - soil system (Hertwig 1931, 1933), the number of load cycles (Tschebotarioff and McAlpin, 1947) and the shear strength during the vibration (Mogami and Kubo, 1953).

Whiffin (1954), Forssblad (1965), and others have pointed out, however, that also the magnitude of the dynamic stresses generated in the soil by the vibrator is of great importance for the results of the compaction.

1. Resonance

According to Hertwig (1931, 1935) the highest density of a soil is obtained at the resonant frequency of the vibrator - soil system where the amplitude reaches a maximum. Also Terzaghi (1943), Bernhard (1952a, 1959), Converse (1954, 1957), and others recommend that the compaction should be done at the resonant frequency.

The resonant frequency varies with the soil type and with the characteristics of the compactor. Sung (1954), Viering (1957) and Lorenz (1960), for example, have pointed out that the resonant frequency depends on the eccentric moment, the weight and dimensions of the vibrator and on the properties of the soil. Also Barkan (1960) and others have discussed the factors affecting the resonant frequency of the vibrator - soil system.

Compaction tests and practical experience do not indicate, however, that vibratory compaction at the resonant frequency will give a higher density than at other frequencies (e.g., Lorenz 1955, 1960; Forssblad, 1955, 1965; Kutzner, 1962). Usually the density of compacted soil will increase with increasing frequency even at frequencies exceeding the natural frequency of the vibrator - soil system. One reason as pointed out by Forssblad (1965) is that the centrifugal force and the momentum and thus the "intensity" of the vibrations also increase with increasing frequency. The resonance effects of the vibrator - soil system will in this case be combined with the effects of an increase of the "intensity" of the vibrations as graphically illustrated in Fig. 5. The resonance effect is in this case "hidden". Due to this reason medium and heavy-weight vibrating rollers often are designed with a frequency just above the resonance frequency of the vibrator - soil system.

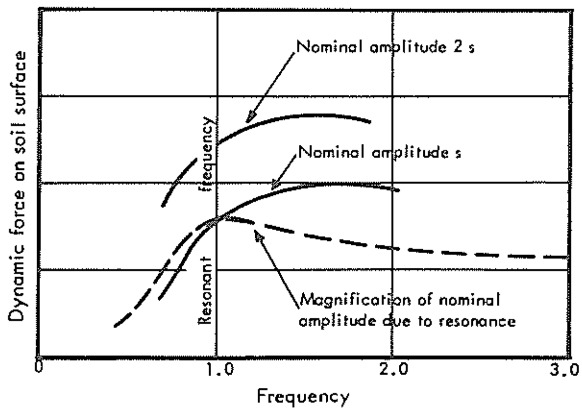


Fig. 5. Effect of Frequency and Amplitude on Vibratory Compaction of Soil

2. Number of Load Cycles

According to Tschebotarioff and McAlpin (1947) the number of load cycles and the centrifugal force govern the density of the compacted soil when the frequency is low. About 10.000 cycles were required to reach the maximum density of a sand when the centrifugal force was 9 and 18.7 kg. The maximum density increased with increasing centrifugal force. When the centrifugal force was 37 and 75 kg, approximately 30.000

load cycles were required. These tests were carried out at frequencies between 1.0 and 25 cps.

Kutzner (1962) also found from vibratory compaction tests with glass beads and dry sand that the density increased with increasing number of load cycles or with increasing vibration time. For the glass beads the increase was small after 500 load cycles. About 9.000 load cycles were required to reach the maximum density of the dry sand at a frequency of 50 cps.

Tests by Whitman and Ortigosa (1968) indicate that the density of a compacted cohesionless soil at accelerations less than 1 g is primarily governed not only by the number of load cycles but also by the stress increase in the soil. The density increased as the logarithm of the number of cycles. The increase of the density was, however, small when the initial relative density was larger than 70% and the ratio of the maximum and the minimum cyclic stress from the vibration was less than 5.

The test results thus indicate that the number of load cycles is of special importance at low accelerations. The number of load cycles required to reach a given relative density is also influenced by the gradation of the soil.

3. Shear Strength During Vibration

Mogami and Kubo (1953) observed that the shear strength of dry sand decreased with increasing acceleration. They investigated two types of dry sand in a direct shear apparatus (Fig. 6) at frequencies ranging from 10 to 50 cps and at accelerations which varied up to 20 g. The test results indicated that the shear strength decreased approximately linearly when the acceleration increased from 0.5 to 1.0 or 2.0 g.

The shear strength of sands under vibratory loading has also been investigated by Linger (1963) with direct shear and triaxial tests. The vibratory tests gave a lower shear strength than the static tests. The superimposed vibratory loads caused a reduction of the normal load for the direct shear tests and of the confining pressure for the triaxial tests.

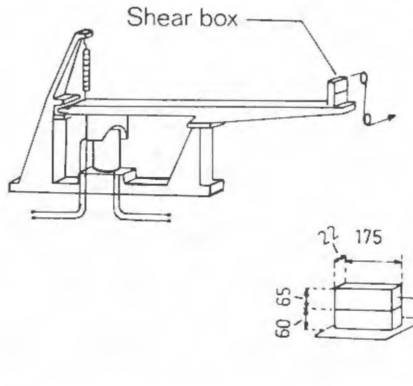


Fig. 6. Test Equipment for Determination of Soil Shear Strength With and Without Vibrations Used by Mogami & Kubo (1953).

The degree of saturation and the water content affect appreciably the compaction properties of sand due to capillary forces between the individual soil particles ("false" cohesion). In Fig. 7 is shown the dry density of uniform coarse, medium and fine sands as a function of the water content (Kutzner, 1962). It can be seen that the dry density of the soil compacted either dry or saturated was approximately the same. The lowest density was obtained when the water content was 5 to 10%. Kutzner also carried out compaction tests where a wetting agent had been added to the pore water. In this case the dry density of the coarse sand after compaction was approximately independent of the water content (Fig. 7). These tests indicate that the compaction of partly saturated cohesionless soils is appreciably affected by capillary forces.

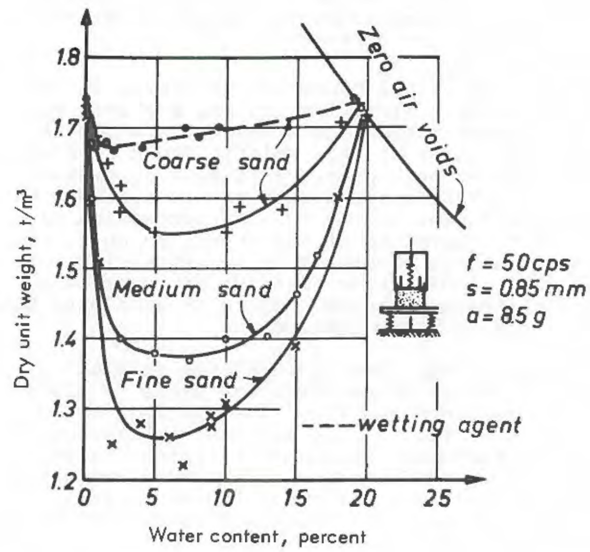


Fig. 7. Vibratory Compaction of Sand at Different Water Contents After Kutzner (1962)

Forssblad (1965) used the vane test to study the shear strength of mainly cohesionless soils with and without vibrations at different water contents. The test data indicated that the vibratory shear strength varied between 1 to 6% of the static shear strength. The lowest value was obtained for gravel, sand, gravelly silt or crushed stone when the soil was either dry or saturated. The vibratory shear strength of all investigated soils was about the same.

4. Dynamic Stresses

The compaction of a soil is to a large extent dependent on the type and the intensity of the stress waves, generated by the vibrator. Longitudinal and transverse waves are transmitted through the soil while Rayleigh and Love waves follow the ground surface. In a transverse wave the soil particles move in a plane perpendicular to the direction of wave propagation while in the longitudinal waves the particles move in the direction of the wave propagation. The transverse waves propagate with about half the velocity of the longitudinal waves (also called compressional waves).

The amplitude of the longitudinal and of the transverse waves generated by vibrating rollers or plate compactors decrease with increasing distance from the vibrator mainly due to geometrical damping. There is also a reduction of the amplitude due to absorption of energy by the soil.

According to Forssblad (1965) the intensity of the stress waves must be high enough to overcome the shear strength of the soil. In a partly saturated sand and gravel with a relatively low apparent ("false") cohesion a vertical dynamic stress of about 0.5 to 1.0 kg/cm^2 was required to reach a relative compaction of 90% with respect to the modified AASHO compaction test. For clays a considerably larger pressure - about 5 kg/cm^2 was required to reach the same relative compaction at the optimum moisture content.

Whiffin (1954) has measured the stress distribution below a vibrating roller, a static roller, a power rammer and a crawler tractor in two types of clayey soils. Whiffin found a relationship between peak pressure (static or dynamic) and the relative compaction of the soil which was independent of the type of compaction machine and of the duration of the stress pulse. The dynamic stress generated by the vibrating roller was about 100% larger than the corresponding static stress increase. These results have been confirmed by other authors.

Forssblad (1965) has measured the vertical dynamic stresses below different types of vibratory compactors and for different soil types. As shown in Fig. 8, the maximum vertical dynamic stress decreased rapidly with increasing depth below the ground surface. Forssblad found that it was possible to calculate approximately the dynamic stress distribution by Boussinesq's equation, for a compacted soil. For a loose soil the dynamic stresses in the soil were less than those calculated by Boussinesq's equation. Also Bernhard (1952a, 1952b, 1962, 1967); Christoffel (1960), and others have measured the stress distribution in the soil below different types of vibratory compactors.

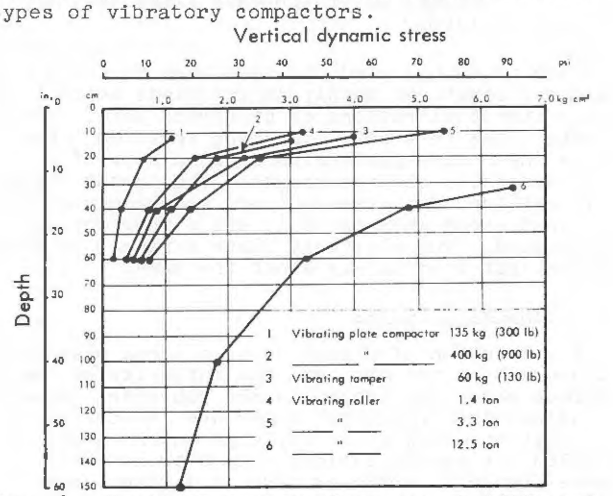


Fig. 8. Vertical Dynamic Stresses at Different Depth Under Vibratory Compactors of Different Types and Sizes According to Forssblad (1965)

Reflection of the stress waves against lower, more compact layers can give special compaction effects. Near such a layer the compaction of a soil will sometimes be better than that close to the surface. Reflection can also have a loosening effect when thin layers of soil are placed on a rigid base.

Luscher et al. (1967); Ortigosa and Whitman (1968) and D'Appolonia et al. (1969) have suggested that both the maximum and minimum dynamic stress in the soil are factors which affect vibratory compaction of cohesionless soils. The minimum dynamic stress in the soil should not be much larger than zero to obtain a good compaction. Even if the minimum dynamic stress is relatively small the soil particles are not free to move.

5. Liquefaction During Compaction

The pore pressure in a saturated sand builds up progressively during the vibration if the relative density is low and the sand is not free-draining. The compactor bogs down when the pore pressure approaches the total overburden pressure of the soil and the vibrator. This phenomenon is called liquefaction. Liquefaction is dependent on the permeability, the initial density of the sand, the number of load cycles, the overburden pressure and the intensity of the cyclic load (Seed and Lee, 1966; Lee and Seed, 1967; Peacock and Seed, 1968; Seed, 1968). The vibratory load and the number of load cycles required for liquefaction increase with increasing relative density and with increasing overburden pressure.

The minimum permeability for the soil to be free-draining is a function of the towing speed, the thickness of the compacted layer and the compressibility of the soil. Approximate calculations indicate that the permeability should be at least 10^{-2} cm/sec. for a layer thickness of 20 cm, a towing speed of 4 km/hour, and a volume compressibility of 0.02 cm^2/kg for the soil to be free-draining and to avoid that the compactor bogs down. It has been assumed that the consolidation ratio should be at least 50% for each pass.

V. VIBRATING ROLLERS

A large number of different types of vibrating rollers are used as shown in Table 1. The frequency varies between 20 and 80 cps and the nominal amplitude between 0.3 and 2.5 mm. Some types of vibrating rollers operate at a higher frequency in combination with a low nominal amplitude. Other types have relatively low frequencies and large amplitudes. The frequency can often be changed by changing the engine speed. There are also vibrating rollers with adjustable eccentric moment so that the nominal amplitude can be varied. Tractor-drawn vibrating rollers usually have a centrifugal force which is 2 to 3 times the total weight of the roller.

The Road Research Laboratory in England initiated in 1945 (and is still conducting) extensive compaction tests with different types of static and vibratory compactors. An early investigation by Hunt (1946) indicated that a higher density was obtained for a coarse sand by a 215 kg vibrating roller than by a 12 ton rubber tired roller or by an 8 ton static smooth wheel roller. The layer thickness was 22 cm. Later compaction tests were carried out on a number of standardized soils (gravel-sand-clay, well-graded sand, uniformly graded fine sand, sandy clay and heavy clay). The first three of these soils contained about 15% of fines. Detailed results from the compaction tests are available in a number of special reports. The test results have been summarized and discussed by Lewis (1954, 1960, 1961, 1967). According to Lewis (1967) vibrating rollers with weights up to 1 ton could only be used to compact granular soils. Tractor-drawn vibrating rollers with weights between 3-3/4 and 8-1/2 tons were suitable for all soil types, also clays. The required number of passes for vibratory rollers were 4 to 6. The maximum layer thicknesses, which were recommended for the test vibratory compactors, are relatively low. The main reason is probably the comparatively high content of fines in the tested soils. Lewis (1961) has published results which show a relationship between the dry density of the compacted soil and the frequency of the vibrations. (Fig. 9). In the tested frequency range the effect of changes in frequency was significant only with the granular soils. (Compare also Fig. 9 with Fig. 5.)

Lewis (1961) has also reported results from compaction tests on a well-graded sand with a 3-3/4 ton tractor-drawn roller at different towing speeds and different number of passes. These test results indicated that a larger towing speed must be compensated by a larger number of passes in order to reach a given relative compaction. It was found that the relative compaction was mainly dependent on the total vibration time on a given surface area.

The test results obtained at the Road Research Laboratory have been discussed and analyzed by Johnson and Sallberg (1960); Morris and Cochrane (1964), and others. Johnson and Sallberg (1960) pointed out that a good correlation was obtained for vibrating rollers between the degree of compaction and the static weight per cm of roll width.

Compaction tests have also been carried out at the National Swedish Road Research Institute (Bruzelius, 1954). Typical Swedish soils (coarse gravel, sand, silt and moraine) were compacted in 0.6 m to 1.0 m lifts. A considerably higher relative compaction was obtained by a 3 ton tractor-drawn vibrating roller and also by a 1.5 ton vibrating plate compactor than by static rollers. The maximum effective compaction depth ranged from 0.5 to 1.0 m for the vibratory compactors. The surface density of the soil was generally lower than the density at some depth below the surface. The maximum dry density was obtained at a depth of 10 to 30 cm.

Two manually-guided vibrating rollers weighing 250 kg and 1.6 tons have been tested at the Central Road Research Laboratory in India. The frequency was one of the factors which was varied during the tests (Gokhale and Rao, 1957).

Garbotz and Theiner (1959) at the Technische Hochschule, Aachen, Germany, have carried out extensive compaction tests with static smooth wheel rollers, vibrating rollers and vibrating plate compactors. Garbotz and Theiner found that uniformly graded cohesionless materials could not be compacted to a higher density by vibrating rollers than by static smooth wheel rollers. The depth effect of vibratory compactors, however, was considerably larger than for static smooth rollers. Vibratory compaction gave the highest degree of compaction for well-graded cohesionless soils.

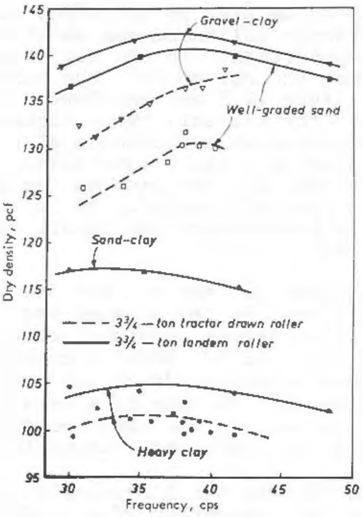


Fig. 9. Relationship Between Dry Density of the Compacted Soils and Frequency of Two Types of Vibrating Roller According to Lewis (1961)

Garbotz and Theiner also found that the degree of compaction of gravel increased when the frequency of a 1.65 ton tractor-drawn vibrating roller was increased from 23 to 37 cps. A further increase in frequency up to 50 cps caused a small decrease of the density. For the compaction of cohesionless soils Garbotz (1964) recommends for vibrating rollers the combination of a relatively high frequency and a low amplitude.

Johnson and Sallberg (1960) have reviewed the factors influencing the field compaction of soils. In this report are summarized results from compaction tests available up to that time.

Forssblad (1965) has published results from compaction tests on sand with 1.4 and 3.3 ton tractor-drawn vibrating rollers and with vibrating plate compactors weighing 40, 120 and 400 kg. The influence of the towing speed and of the number of passes was investigated for the 3.3 ton tractor-drawn roller. The required number of passes to reach a specified density increased with increasing towing velocity. Test results indicated that the surface capacity was approximately independent of the rolling speed between 3 and 6 km/h. Lower speeds than normal are recommended when the layer thickness is large and when a very high degree of compaction is required.

Tynan and Morris (1968) have investigated six types of tractor-drawn vibrating rollers weighing from 3.2 to 9.9 tons. The compaction tests were carried out on crushed rock, uniform dune sand and on medium clay at different water contents. The layer thickness was about 25 cm before the compaction. In the weight class 3.2 to 5 tons, the roller with the largest nominal amplitude (1.7 mm) and lowest frequency (24 cps) gave for all soil types higher densities than the rollers with a frequency of about 40 cps and a relatively small amplitude. For crushed rock and clay the density increased with increasing number of passes up to 16. For the dune sand the increase of the density was small after 2 passes.

The optimum water content for the field compaction tests was for the crushed rock 1 to 2% higher than the optimum moisture content, determined by the modified AASHO compaction test. No significant crushing effect was observed for this material. For the dune sand the optimum moisture content was the same for the field as for the laboratory compaction tests.

D'Appolonia, Whitman and D'Appolonia (1969) have investigated the compaction of a uniform dune sand by a 5 ton and by a 3 ton vibrating roller. The layer thickness was large. Two passes of the 5 ton roller gave the specified compaction (75% relative density) down to a depth of about 60 cm. The compaction was not sufficient, however, close to the surface down to a depth of about 25 cm. The density at depths larger than about 25 cm increased substantially with increasing number of passes. D'Appolonia et al. recognized three different compaction zones. The soil near the surface was considered to be "overcompacted". Below this zone to a depth of 60 cm the compaction was believed to be determined by the maximum and the minimum dynamic stress generated in the soil by the roller. The minimum dynamic stress must be low enough to allow the particles to move. The compaction of the soil below a depth of about 60 cm was considered to be caused mainly by the repetitions of stress (number of load cycles).

The problem with a low surface density of uniform cohesionless soils has also been discussed by Bruzelius (1954); Garbotz and Theiner (1959); Wiklund (1960); Forssblad (1965), and others. "Overcompaction" can be caused by heavy static rollers as well as by medium and heavy weight vibrating rollers and plate compactors. Heavy vibrating rollers can cause "overcompaction" down to a depth of 40 to 50 cm as indicated by Baker and Moorhouse (1968). The best surface compaction is obtained with light vibrating rollers, light and medium weight plate compactors, or with light and medium weight static rollers. To increase the surface density it is also advantageous to compact the soil at a high water content.

Tynan and Morris (1968) observed that the surface density of a well-graded crushed rock was low particularly when a heavy, high frequency roller was used, and the compaction water content was low. In this case reflection of stress waves against the underlying rigid subbase could have contributed to the observed effect.

Hall (1968) carried out compaction tests on crushed limestone, sand and lean clay. The sand contained 8% of material smaller than 0.06 mm. The following types of rollers were investigated:

a. Tractor-drawn vibrating roller, weight 3.2 tons, low frequency.

b. Tractor-drawn vibrating roller, weight 1.4 tons, high frequency.

c. Self-propelled vibrating roller, weight 2.4 tons, low frequency.

d. Rubber-tired static roller, weight 50 tons.

Rollers a and d gave for the crushed limestone and the clay the highest relative compaction. The compaction by the two rollers was similar. On sand compactor c gave the best overall results. Roller b gave satisfactory results only on the sand.

The relationship between technical data and the performance of vibratory rollers has been discussed by Tope (1967).

Heavy vibrating rollers are often used to compact rock fill and granular soils containing large stones and boulders. Field compaction tests indicate that rock fill can be efficiently compacted in thicker layers than sand and gravel. The large depth effects which are obtained with vibrating rollers on rock fill is probably due to the high impact forces which are generated when the vibrating drum comes in contact with large stones. Also the damping in rock fill is relatively low.

Vibratory compaction of rock fill usually causes a reduction of the volume by 5 to 7%. Field experience indicates that 10 to 15 ton vibrating rollers with 5 to 10 passes can compact rock fill in 1.0 to 2.0 m lifts. Vibrating rollers with weights between 3 and 5 tons are used to compact rock fill in 0.5 to 1.0 m lifts.

Heavy vibrating rollers are also used to compact natural sand deposits with a low initial density in order to increase the bearing capacity and to reduce the settlements. The sand may either be fully or partly saturated. If the depth is relatively large, up to 20 to 30 passes may be required. Borings and other measurements have indicated a maximum depth effect of 2.5 to 3 m for 10 to 15 ton vibrating rollers (Baker and Moorhouse, 1968).

Winter compaction of soil and rock fill has been investigated in Canada, Sweden, Norway, Finland, USA and other countries. Frozen soils except clean and dry gravel, crushed rock and rock fill can generally not be compacted to the same dry density as unfrozen soils. The difficulties with winter compaction increase with increasing water content and with decreasing grain size and temperature (Bernell, 1965). Vibrating sheepsfoot rollers have been used successfully to compact gravel during the winter. Due to the high contact pressures lumps of frozen soil can be crushed. Test results indicate, however, that also with this method the dry density of compacted gravel is about 10% lower than that of the same soil compacted during the summer (Bernell, 1965).

During the winter unfrozen soil can often be obtained from excavations. When the unfrozen soil is spread out in cold weather the surface freezes rapidly. It is therefore important to compact the soil immediately after it has been placed.

VI. VIBRATING PLATE COMPACTORS

The most commonly used plate compactors weigh between 100 and 500 kg. With two eccentrics working synchronously it is possible to obtain a directed vibratory force and to move the vibrator during the compaction. By adjusting the eccentric element the velocity and the direction of motion can be varied. A plate vibrator can also be moved with only one rotating eccentric if the eccentric is placed at the front end of the bottomplate.

The frequency of heavy self-propelled vibrating plate compactors is usually between 12 and 20 cps while the frequency of small vibrating plate compactors can be as high as 70 to 80 cps (Table I). The amplitudes vary between 0.5 and 5 mm. The centrifugal force is usually 5 to 10 times the total static weight of the vibrator.

The action of plate vibrators have been reviewed by Converse (1954); Bernhard and Finelli (1954); Lorentz (1960), and others. Siedenburg (1957) has calculated for inelastic soils the motion of a vibrating plate compactor without a spring-supported engine bed plate. Calculations for plate compactors with spring-supported engine bed plates have been made by Bathelt (1956), and by Bathelt and Kock (1956). Bathelt used a mechanical model consisting of three springs to analyze the behavior. Two of the springs could not expand after they once have been compressed. The calculations showed that the centrifugal force for a self-propelled vibrator should be between three to ten times the total weight of the vibrator. Similar calculations have also been carried out by Ephremidis (1959) and Weber (1965).

Also Jurecka (1965) has calculated the motion of vibratory compactors and the energy distribution in the soil.

Compaction tests with vibrating plate compactors have been carried out in Germany during the 1930's as mentioned in the introduction (Müller and Ramspeck, 1935; and Ramspeck, 1937).

Bernhard (1952a) has carried out compaction tests with a 1.5 ton vibrating plate compactor and a multiple-plate compactor. A relative compaction of 95% with respect to the standard AASHO compaction test was obtained for a uniform silty sand by the 1.5 ton vibrator down to a depth of 75 cm.

Converse (1952, 1954) has investigated the compaction of a uniform beach sand by a vibrating plate compactor with a weight of about 5 tons. The dry density of the beach sand was at least 95% with respect to the modified AASHO compaction test down to a depth of 0.45 to 0.60 m and 93 to 96% between 0.60 to 1.50 m below the ground surface.

Greenman (1948) has investigated the vibratory compaction of sand and gravel. The soil could be compacted satisfactorily to a depth of 30 cm with a light plate compactor. The relative compaction was higher than that which could be obtained by internal vibrators or by pneumatically operated tampers.

Aldous and Wills (1952) have investigated the vibratory compaction of crushed rock, gravel and sand. The soils were compacted in 15 to 30 cm lifts by vibrating plate compactors weighing either 70 or 170 kg. The maximum density of the gravel was reached after two passes. The unit weight of the soil in the lower half of the 30 cm lifts was less than in the upper half.

Plantema (1954) used a 1.5 ton vibrating plate compactor to compact a 5 meter thick sand fill. The highest relative compaction was obtained at a frequency of 20 cps. The compaction was checked by soundings with a Dutch cone penetrometer. An appreciable increase of the penetration resistance was observed after the compaction down to a depth of 2.0 m. The velocity did not affect appreciably the compaction. With a light compactor the best compaction was obtained at a velocity of 4 m/min.

Compaction tests with 1.48 ton, 140 kg and 40 kg plate compactors have been carried out by Streck and Schmidtbauer (1954). It was possible to compact a sand to a relative density of 85% within a depth of 60 to 80 cm with the 1.48 ton plate compactor, within a depth of 20 to 30 cm with the 140 kg compactor and within a depth of 20 to 25 cm with the 40 kg compactor.

Weiss (1956) has investigated the compaction of sandy gravel and gravel with a 7.8 ton vibrating plate compactor. A relative density exceeding 50% with respect to the modified Proctor compaction test was obtained down to a depth of 1.2 m for the sandy gravel and 1.7 m for the gravel.

Schaeffer (1958) has investigated the effects of frequency and velocity for a light vibrating plate compactor on the compaction of sandy gravel. The frequencies 25, 33 and 40 cps and the velocities 4.3m/min, 7.4 m/min., and 10.5 m/min. were investigated. The highest density was obtained with the lowest frequency and with the lowest velocity.

Garbotz and Theiner (1959) tested a 2.25 ton vibrating plate compactor. The most favorable frequency range was 12 to 17 cps.

Garbotz and Christoffel (1964) have carried out tests with vibrating plate compactors and vibrating rollers on crushed rock (ballast material).

Kronenberger (1960) has investigated the compaction of vibrating plate compactors without spring supported engine bed plates. The vibrating characteristics and the weight of the compactors were varied. The maximum dry unit weight and the maximum equivalent modulus of elasticity of the compacted soil were reached at a frequency of about 40 cps. Also Lammlein (1953) and Weber (1965) have investigated the performance of vibrating plate compactors. Compaction of sand by plate compactors has also been discussed by Hoppman (1962).

The Road Research Laboratory in England has made comprehensive investigations of vibrating plate compactors (Lewis, 1961). The depth to which soils could be compacted increased with increasing weight and with increasing contact area of the vibrators. It was possible to compact satisfactorily a well graded sand to a depth of 30 to 45 cm with a 1.5 ton or a 2 ton plate compactor. The depth effect at the same weight was larger for the plate compactors than for the vibrating rollers. Lewis found also that fewer passes were required for a plate compactor than for a vibrating roller to reach a given density. The surface and the volume capacities given by Lewis (1967) for plate compactors were, however, much lower than those for vibrating rollers.

Forssblad (1965) has reported results from extensive series of compaction tests with vibrating rollers and plate compactors. A relative compaction of at least 90% with respect to the modified AASHO compaction test was reached for the layer thicknesses given in Table 4.

TABLE 4. -- NORMAL LAYER THICKNESS FOR VIBRATION OF COHESIONLESS SOILS WITH LESS THAN 5 TO 10% OF THE MATERIAL SMALLER THAN 0.06 MM

Vibrating plate compactor, 100 to 200 kg	20 cm
Vibrating roller, 1 to 2 tons	30 cm
Vibrating plate compactor, 400 to 600 kg	40 cm
Vibrating roller, tractor drawn, 3 to 4 tons	50 cm

Tests and practical experience indicate that it is possible to compact sand and gravel to a high relative density even in high lifts. Also relatively fine sand with an average grain size between 0.06 and 0.2 mm can efficiently be compacted in high lifts by vibrating rollers or plate compactors. However, comparatively small amounts of fines (silt or clay size particles) will increase considerably the compaction effort required to reach a certain relative compaction. Also the layer thickness must be decreased with increasing content of fines.

VII. VIBROFLOTATION

Vibroflotation has been used since about 1935 to compact saturated,cohesionless soils. The method has been described in a number of publications (e.g., Schneider, 1938; Steuerman, 1939, 1948; Tschebotarioff, 1946; Schneider, 1948; Rappert, 1952; Cassel, 1956; D'Appolonia et al, 1953; Mobus, 1959; Watanabe, 1963; Thorburn and MacVicar, 1968).

Vibroflotation has been developed and applied by the firms J. Keller, Germany; Vibroflotation Foundation Co., USA; Cementation Co., England. The method has also been used in USSR and Japan. Projects where vibroflotation has been used have been described by Schneidig (1940); Leussink (1948); Fruhauf (1949); D'Appolonia (1954); D'Appolonia and Miller (1955); Plannerer (1965); Hansbo et al (1968); and by Doscher (1968).

The especially designed large internal vibrators (vibrofloats) have generally a diameter of 37.5 cm and operate at a frequency of 30 cps. The nominal amplitude is about 10 mm and the centrifugal force is up to 10 tons. The vibrators are provided with jetting devices which facilitate the insertion and the withdrawal of the vibrators. Sand or gravel is added during the vibration to compensate for the subsidence at the ground surface as the soil around the vibrofloats is compacted. With this method the soil can be compacted down to a depth of 20 to 30 m. Vibroflotation has mainly been used to compact natural strata of sandy soils with a relatively high permeability. A maximum amount of 8 to 16% of particles smaller than 0.06 mm can generally be allowed.

Test results by D'Appolonia et al. (1953) indicated that the relative density of a fine uniform sand is at least 70% to a distance of 1 m from the vibrofloats. A spacing of 2 m between the insertion points gave a minimum relative density of 70% throughout the compacted area. The distance from the vibrofloats to which the soil can be compacted appears to increase with increasing grain size and with increasing uniformity of the soil.

Abu-Wata and Said (1958) used vibroflotation to compact a uniform dune sand which had been placed under water. The spacing of the insertion points varied between 1.65 and 2.5 m. The compaction of the soil was checked by soundings with a Dutch cone penetrometer. A substantial increase in the measured penetration resistance was observed close to the insertion points. The average initial penetration resistance before the compaction was about 60 kg/cm^2 while the minimum penetration resistance after compaction at a spacing of the vibrofloats of 2.5 m was 120 kg/cm^2 below a depth of 4.5 m. The penetration resistance increased with decreasing spacing of the vibrofloats. The increase in penetration resistance and of the compaction was small close to the surface.

Scheelhaase (1959) has described a project where vibroflotation was used to compact a uniform coarse to fine sand for a turbine foundation. The compaction was checked by static penetration tests. The point resistance varied between 0 to 150 kg/cm^2 before the compaction and between 250 and 500 kg/cm^2 after the compaction. The penetration tests indicated that the soil was compacted to a distance of 1.0 to 1.1 m from the insertion points.

Hansbo et al. (1968) has used the vibroflotation method to compact a uniform sand with an average grain size of 0.5 mm to a depth of 8 to 10 m. The compaction was determined by pressiometer tests, plate load tests and penetrometer tests. A substantial increase of the penetration resistance was observed to a distance of 0.6 to 0.7 m from the insertion points.

Internal vibrators which are normally intended for concrete have also been used to compact saturated cohesionless soils (Wenner and Saare, 1954; and Bureau of Reclamation, 1963). According to Wenner and Saare (1954) sand and gravel can be compacted to 90 to 100% with respect to the modified AASHO compaction test to a distance of 0.25 to 0.50 m from the insertion points.

Forssblad (1965) has reported results from compaction tests on sand and gravel with internal concrete vibrators. The size of the vibrator, the frequency and the amplitude were varied. A density between 90 and 100% of the modified AASHO maximum dry density was obtained. The radius of action of the vibrators was found to be smaller, and the required vibration time was found to be longer than in concrete.

The vibration of sand, silt, loess and clay with internal vibrators has been investigated theoretically and experimentally by Poteur (1968).

VIII. LABORATORY COMPACTION TESTS

Laboratory compaction of cohesionless soils by vibration has been studied by many. The soil has, in the laboratory tests, usually been compacted in a Proctor mould placed on a vibrating table. The influence of frequency, amplitude and acceleration of the vibrating table as well as the influence of surcharge load, soil gradation and moisture content have been studied.

Schäffner (1962) has investigated the vibratory compaction of dry sand. Typical test results are shown in Fig. 10. It can be seen that the void ratio decreased with increasing acceleration. The highest density was obtained at frequencies up to 100 cps, probably due to the very small amplitude at high frequencies. Schnaffner has derived an empirical equation which relates the void ratio of a sand with frequency and amplitude.

Kutzner (1962) has investigated the vibratory compaction of glass beads. The experiments were carried out on a vibrating table at frequencies which varied between 5 and 100 cps. Test results are shown in Fig. 11. At frequencies between 40 and 100 cps the density increased rapidly as the acceleration approached 1 g. A further increase of the acceleration did not cause a further increase of the density. The large amplitudes combined with impacts at low frequencies may explain the relatively good compaction within this frequency range.

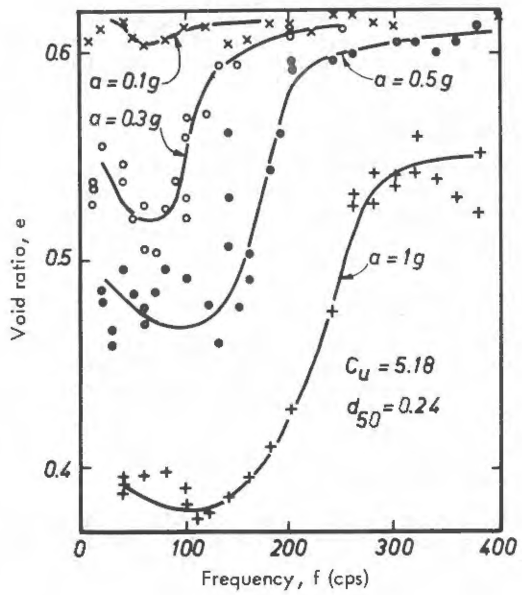


Fig. 10. Compaction of Dry Coarse Sand After Schäffner (1962)

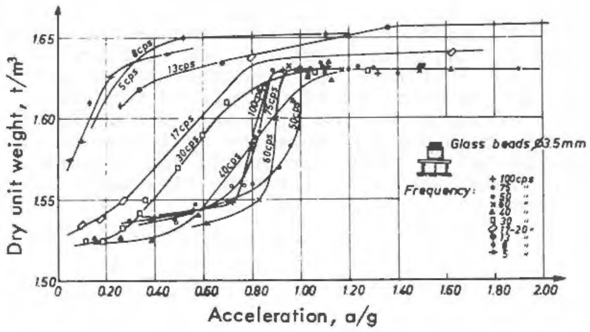


Fig. 11. Influence of Acceleration on the Vibratory Compaction of Glass Beads After Kutzner (1962)

Selig (1963) has studied the effects of frequency, acceleration and surcharge load on the vibratory compaction of a uniform dry sand. Test results indicated that the density increase was small (less than 1%) after two minutes of vibration. The maximum density was obtained at an acceleration between 1 to 2 g. The acceleration required to reach a certain density increased with increasing overburden pressure.

Ortigosa (1968) and Whitman and Ortigosa (1968) found from vibratory compaction tests that the acceleration without surcharge should for a dry fine sand be at least 1 g. The maximum density was obtained at 2 g. When the acceleration was larger than 2 g the density decreased due to "overcompaction". With surcharge load the required acceleration was 1 to 3 g depending on the intensity of the overburden pressure. Also Dunglas (1967) has investigated the vibratory compaction of sand. Dunglas used a special test mould where water pressure was used as surcharge load.

Forssblad (1965) found that the dry unit weight increased for a dry sand with increasing acceleration up to 2 g, and up to 3 g when the same sand was compacted partially or fully saturated. The dry unit weight was approximately constant when the acceleration exceeded these limiting values. No surcharge was used during the experiments. Forssblad also found that the dry unit weight of the compacted soil was independent of the direction of the vibrations.

Laboratory compaction tests on sand, crushed limestone and crushed quartzite have also been carried out by Gomes and Graves (1962) and by Honigs, Valente and Graves (1963). These materials were compacted in a Proctor mould by a surface vibrator consisting of a loud-speaker attached to an aluminum plate. The frequency could be varied. Test results indicated that the stress waves generated by the loud-speaker affected the compaction of the soil. The maximum density occurred at some distance below the soil surface.

Standardized vibratory compaction tests have been mentioned in the Chapter "Elements of soil compaction". The development of test procedures have been discussed by e.g., Kolbuszewski (1948); Burmister (1950); Pauls and Goode (1950); Felt (1958); Leussink and Kutzner (1962); Johnson and Sallberg (1962) and by Pettibone and Hardin (1964).

A vibrating table and a surcharge load or a vibrating tamper or hammer are used in the standardized vibratory test methods, as illustrated in Fig. 1. Pettibone and Hardin (1964) have found for the first mentioned method that the density of the compacted soil at a frequency of 60 cps increased with increased amplitude of the vibrating table. The increase in density was insignificant when the vibration time was longer than 6 minutes.

IX. DAMAGE CAUSED BY VIBRATIONS

Vibratory soil compactors as well as blasting, pile driving and heavy road traffic may damage adjacent structures. Surface waves generated by medium and heavy weight vibratory compactors can often by observed at large distances from the vibrator. Since also vibrations of very low intensity can be felt by man, the risk of damage to buildings and other structures is often exaggerated.

Measurements indicate that the damage is mainly dependent on the maximum velocity of the vibrations (Fig. 3). The risk of damaging adjacent buildings is small if this velocity is less than 0.3 to 1.0 cm/s as indicated by Scheelhaase (1962); Susstrunk (1959); Reiher and Soden (1961) and others. The risk of damage is also dependent on the quality of the building and the type of the foundation.

Ground oscillations from vibratory compactors have been measured by Susstrunk (1959) and Forssblad (1965). An oscillation velocity of 1.0 cm/sec was observed by Forssblad at a maximum distance of 5.5 m from a 3.3 ton tractor-drawn vibrating roller. There were large variations of the ground oscillations due to differences in the soil conditions. Additional investigations will be of interest.

Due to large variations in the soil conditions, in the quality of buildings and in size and type of the compactors it is difficult to establish general rules about the risk of damage by vibratory compactors. This risk must, however, be carefully considered, especially when heavy vibratory compactors are used.

Buried pipes can also be damaged during vibratory compaction of trench-fills. Minimum thickness of the soil layer above the pipes during compaction by vibration or tamping are given in German specifications (Forschungsgesellschaft für das Strassenwesen, 1964).

Retaining walls, abutments and basement walls are sometimes damaged by the high lateral pressures which develop during the compaction of the backfill. Measurements indicate that these pressures can be several times larger than those used in the design (Sowers et al, 1957; Broms, 1967; and Broms and Ingelson, 1967). Also the lateral earth pressures which remain after the compaction are often high and can approach the Rankine passive earth pressure close to the ground surface. Further studies are recommended.

X. FURTHER RESEARCH

This report on the state-of-the-art of vibratory soil compaction indicates that there is need for further research in this field. The following important points can be mentioned:

1. Compaction Mechanism

Transmission of the vibrations from vibrator to underlying or surrounding soil. Propagation of stress waves in the soil. Damping properties of soils at different frequencies, amplitudes, gradations, water contents, etc.

2. Special Features of Vibratory Compaction

Rational relationships between suitable layer thickness and number of passes for different types of soils and vibratory compactors. Crushing of soil and rock particles during vibratory compaction. Winter compaction of soil and rock fill.

3. Compaction Control

Development of efficient and rapid methods for compaction control of thick layers and for soils containing large stones. Development of improved laboratory compaction methods based on vibrations.

4. Damage to Structures

Damage to buildings by surface oscillations produced by vibratory compactors. Lateral earth pressures on retaining walls, abutment and basement walls during and after vibratory compaction of soils.

5. Technical Data

Recommendations about the technical data of vibratory compactors which should be included in specifications, technical reports, etc.

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