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**NORWEGIAN EXPERIENCE WITH STEEL PILE
FOUNDATIONS TO ROCK***

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SYNOPSIS

THE paper reviews 25 years of Norwegian experience in the use of steel piles for foundations. The condition of soft clay overlying a hard and often steeply inclined rock surface which is common in Norway creates a number of problems in the design and driving of point bearing piles to bedrock. Methods of pile driving which have been evolved to ensure a satisfactory sealing of the pile point into rock are described in detail.

The problems of the buckling of piles under load and of the corrosion of steel piles in the ground are discussed in the light of recorded experience. Finally, the settlements of single piles and of buildings are discussed. The settlement of a successful single pile is found approximately equal to the elastic compression of the pile, whereas observations indicate that the settlements of buildings are two to four times this value.

INTRODUCTION

In 1930 foresighted engineers in Oslo—in particular Mr. O. Folkestad and Mr. F. Selmer—realized the significance of steel piles for foundations under Norwegian soil conditions, and after careful consideration of the new problems involved, they used steel piles under a 14-story monumental building, one of the first tall buildings

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in Oslo. Since then the application of steel piles has developed so rapidly that today it is the most frequently used type of pile in Oslo.

Steel piles have found extensive application in Norway due to their unquestionable superiority under the special Norwegian soil conditions. In the most populated eastern and southern districts of Norway the bedrock is either exposed or it is reached at reasonable depths. The rock is in general an extremely hard granite, but it is now and then penetrated by layers of shale. As a result of the regional movements during the Tertiary period and of the action of the glaciers of the last ice age, the surface of the rock is extremely irregular with hills and deep canyons. During the last part of the ice age the country was depressed under the weight of the glaciers and the deeper part of the rock surface was covered by a soft marine clay. Due to the elevation of the land, which followed the withdrawal of the glaciers, the clay deposits came above seawater level and these formations today form the subsoil of the most populated areas in Norway.

From a foundation point of view, it is a characteristic feature of these formations that the clay is very soft and the depth to rock is extremely variable. This means that the length of the piles will often vary so much that precast concrete piles cannot be used. Steel piles can be elongated by welding and therefore they can, with a minimum of loss due to cutting, be driven to the individual lengths required at the different parts of a site. Another problem is the displacement and remoulding of the soft and sensitive clay due to the penetration of the piles. If the volume of the piles is large, the remoulding may lead to an essential reduction in the shear strength of the clay and the displacement may cause a serious upheaval of the previously driven piles and damage to neighbouring buildings. Steel piles are superior in this respect to all other types of piles since the displacement is 10 and 20 times smaller than that of concrete and timber piles respectively.

There are thus several technical advantages connected with the use of steel piles in Norway, and since the price is almost the same as for concrete and timber piles calculated per unit length and per ton allowable load, it will be readily understood why steel piles are preferred for the foundation of all important buildings.

In connection with the use of steel piles a number of special problems arise. There are the questions of how to select the most

suitable cross section of the pile, how to construct the point for ensuring a good bearing on rock, what loads can be permitted and what settlements can be expected. Of special interest are the questions of corrosion and the danger of buckling of piles in soft clay.

These problems were carefully considered in connection with the first use of steel piles in Norway (Folkestad 1934, F. Selmer 1938). Since then only a few systematical investigations have been made, but many experiences have been collected and a tradition has been established in the design and driving of steel piles. 1956 marks the 25th anniversary of the use of steel piles in supporting Norwegian buildings. The author has therefore found it useful to take this opportunity to collect some of the experiences derived from the use of steel piles in Norway during the past 25 years.

DESIGN OF PILE AND PILE POINT

The cross sections of the steel piles which were used for the first buildings in Oslo are shown in Fig. 1. The final design was a pile made by welding two angles together to form a box section which was covered by a layer of mortar in order to prevent corrosion. Since it proved difficult to keep the mortar intact during the driving of the piles, the use of unprotected piles was suggested. In this connection the building authority of Oslo made an investigation of the corrosion of some old steel piles, and they came to the conclusion that unprotected piles could be used in clay if the clay showed a basic reaction, expressed by the criterion that the pH be higher than 7.

Since then only unprotected steel piles have been used, and the H-sections have found an extensive application. Fig. 2 shows the most commonly used sizes and gives the specification of pile caps as required by the authorities of Oslo.

Another type of pile which has been used in many cases is the box pile made by welding two angles together (Fig. 3). This pile has the advantage of being relatively stiff, and under difficult driving conditions it is easier to ensure the straightness of this pile than of the more slender types. Moreover, the central hole permits an inspection of the straightness of the pile and of the welds after driving. After inspection the hole is filled with concrete. Under difficult foundation conditions in corrosive ground and with great depths to rock this type has been preferred to the H-piles.

Even the first piles, which were driven in 1931, were supplied with a special point in order to safeguard against sliding of the pile on an inclined rock surface. Careful considerations were given to the form, and the final solution was to make the point out of a

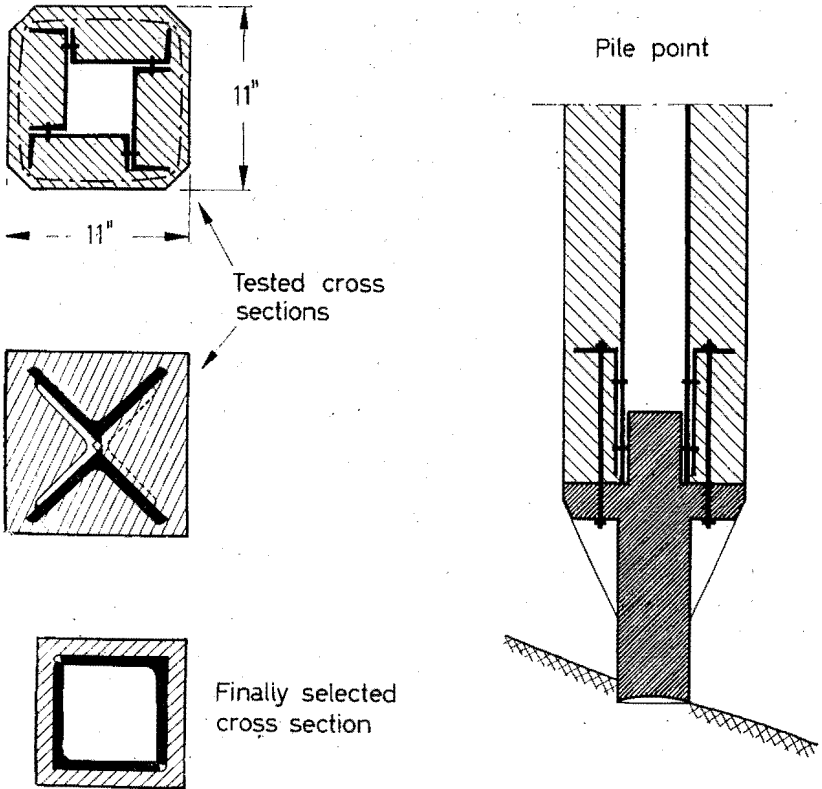


FIG. 1.—CROSS SECTIONS OF STEEL PILES AND PILE POINT USED FOR FOLKETEATERET OSLO 1931. (FOLKESTAD 1934).

round steel bar, the lower end of which was hollow ground. The sharp edges of the bar should in this way be able to secure a hold in the rock immediately after the contact. The form of the bar should, moreover, facilitate the chiseling of a contact area in the rock below the point. In Scandinavia, this type of pile point is called an "Oslo-point".

The "Oslo-point" has been used on the majority of steel piles

driven in Norway and Sweden since 1931. The first points used for the box piles were made of cast iron. For the H-piles the points are made of tempered steel bars of 3 or 4 inches diameter. A slice is cut into the web of the H-section and the bar is welded to the sec-

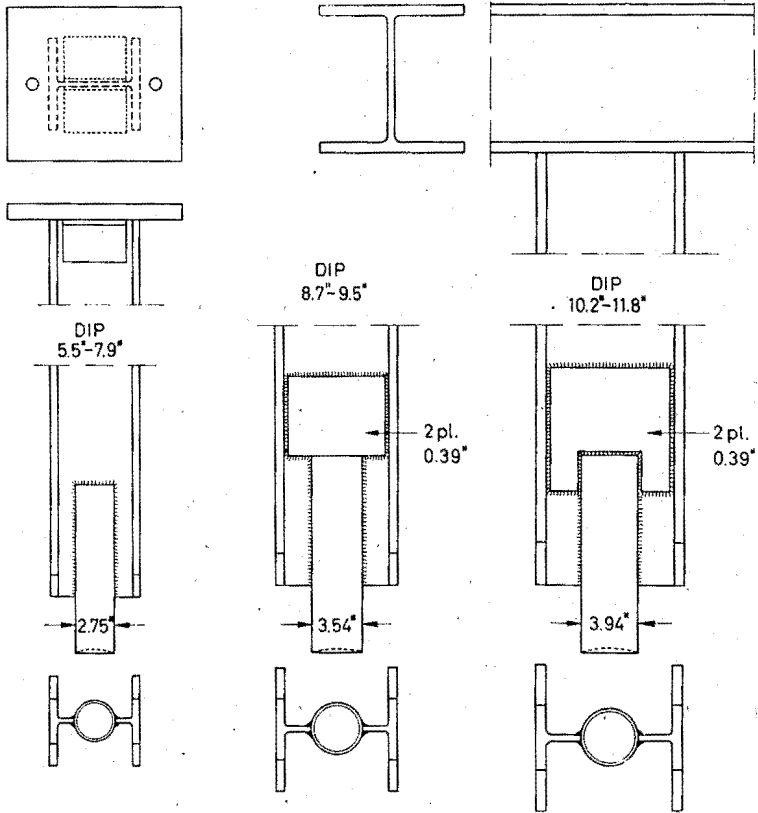


FIG. 2.—H-SECTION STEEL PILES WITH POINTS AND CAPS AS REQUIRED BY THE BUILDING AUTHORITY OF OSLO.

DIP is the notation of a wide-flanged H-beam in the Deutsche Industrie-Normen (DIN 1025).

tion. Fig. 3 shows the connection between point and pile and the details of reinforcing the connection for heavier sections. Fig. 4 is a photograph of a pile point for an H-pile and Fig. 8 a point for a box pile.

Recently a detailed investigation of steel piles has been made.

The results show that, for the hard rock encountered in Norway, the hollow ground steel bar is the right solution for the pile points. The best results have been obtained by hardening the lowest 4 inches of the bar until its Brinell hardness is between 400 and 600.

The first steel piles used in Norway were designed for the unit

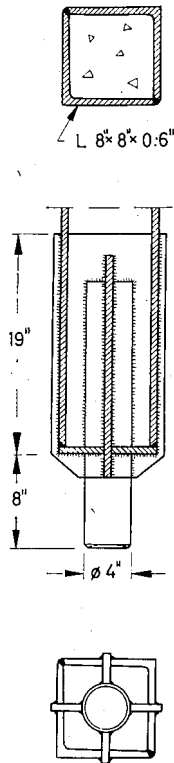


FIG. 3.—BOX PILE USED UNDER DIFFICULT FOUNDATION CONDITIONS.

stress 17100 lb/in². These piles were protected against corrosion by a layer of mortar. With the use of unprotected piles the Building Authority of Oslo reduced the design loads on steel piles. Today, stresses up to 14200 lb/in² are permitted for piles with a length smaller than 40 feet. For pile lengths from 40 to 100 feet, 11400 lb/in² is allowed and for piles which are longer than 100 feet the allowable unit stress is reduced to 9200 lb/in².

DRIVING OF STEEL PILES TO ROCK

In the driving of steel piles to rock one can obviously distinguish between two stages, (1) the driving of the pile through the clay until contact with rock, and (2) the securing of the pile point into the rock.

The purpose of the first stage is simply to drive the pile at the right location, with the right inclination and in such a way that the pile is as straight as possible after the driving. In soft clay some difficulties may be encountered. The Norwegian Geotechnical In-

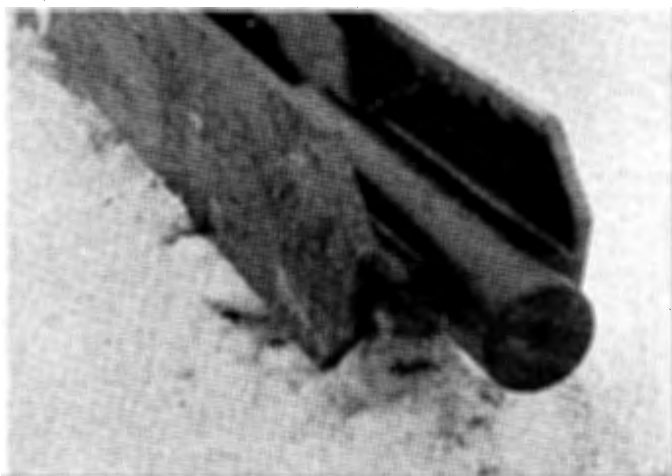


FIG. 4.—PHOTOGRAPH OF PILE POINT FOR H-SECTION.

stitute has studied the driving of slender piles and determined those factors which may contribute to misalignments of the piles. Some of these factors are illustrated in Fig. 5.

The purpose of the second stage of the pile driving is to work out in the rock surface a satisfactory contact area which will ensure a transmission of the static pile load to the rock without excessive stresses in the steel point or in the rock. It should be remembered that in many cases the rock surface is very hard and has such an inclination that friction between steel and rock is not sufficient to prevent a sliding of the pile point. A detailed study of this question has indicated that the procedure which should be used is as follows:

When, during the driving of the pile, the point is approaching

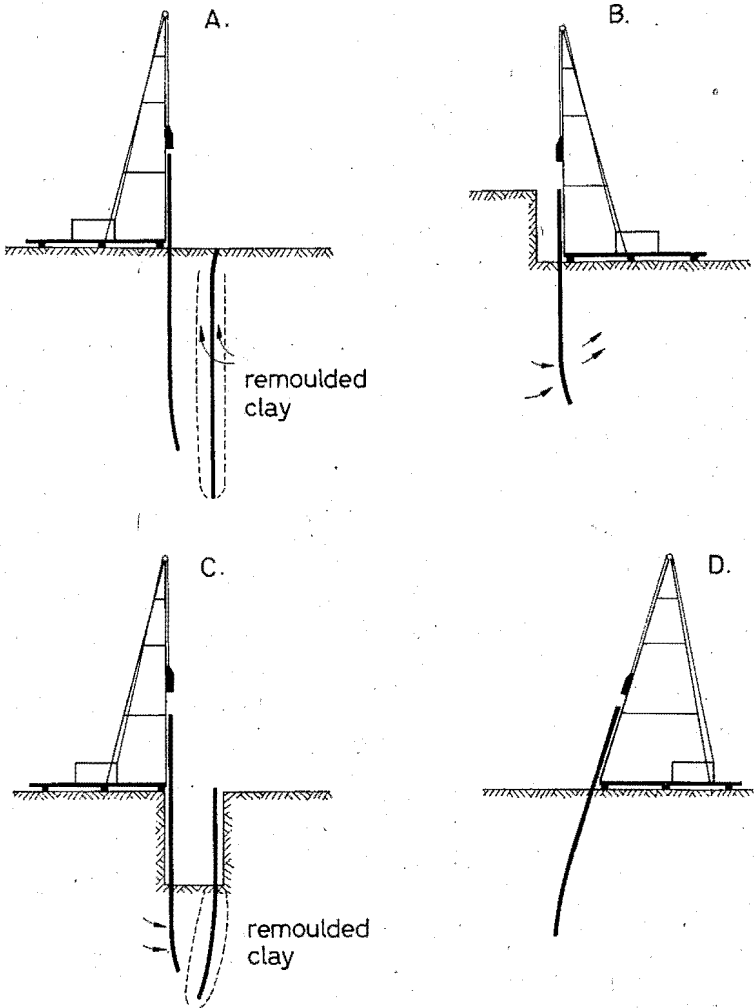


FIG. 5.—FACTORS WHICH CAUSE MISALIGNMENTS OF PILES DRIVEN INTO SOFT CLAY.

- A. The pile will have a tendency to bend in the direction of newly driven neighbouring piles around which the shear strength of the clay is previously reduced.
- B. Along the walls of an excavation the displacement will mainly take place in direction of the excavation and the pile will tend to bend away from the wall.
- C. In trenches and shafts the displaced clay will primarily be squeezed up in the bottom of the excavation and the piles will consequently have a tendency of a curvature against the centerline of the excavation.
- D. Due to their weight batter piles will tend to bend in a downwards direction.

the rock surface, the fall of the ram should be reduced as much as possible and the pile driving should be continued with great care in order to prevent a sliding of the point when it comes in contact with the rock surface.

When the pile point has come into contact with the rock surface—which under Norwegian conditions is at once recognized by a characteristic change in the response of the ram—the driving is stopped and the height of the pile is measured by levelling.

The pile driving is then continued using very small drops of the ram. It is recommended that, on hard and inclined rock, falls of only 6-8 inches be used at the start. After the pile has been subjected to a sufficient series, each of 20 blows, that the penetration of the pile has practically stopped, the fall is increased to double the height.

By the great number of blows used in this procedure it is attempted to chisel the point into the rock in a similar way to the drilling of holes into rock with compressed air equipment.

The driving is stopped when a satisfactory contact area has been established. This can be controlled by a so-called deformation measurement (Glanville, Grime and Fox 1938). By drawing a guided pencil horizontally over a piece of paper fastened to the pile during a single blow of the ram a graph is obtained from which the elastic compression of the pile is easily measured. From Hooke's law the average stresses in the pile can be calculated. In this way it is possible to determine the fall of the ram which sets up stresses in the pile of the order of, for instance, 1.5 times the static load which the pile has to carry in the final construction. If the pile is driven sufficiently hard to withstand for instance 10 blows with this fall without permanent set, this will ensure that a satisfactory contact between pile point and rock has been established (Vold 1956).

Even if the displacement resulting from the driving of steel piles through clay is relatively small, there may still be a danger of upheaval of a pile during the driving of neighbouring piles. It is, therefore, a general practice in Norway today to measure the height of all piles by levelling immediately after the driving. The levelling is repeated after all piles are driven and if some of the piles show an upheaval they are redriven.

In order to illustrate the importance of selecting a correct driv-

ing procedure and of supplying the piles with a point if hard rock is encountered, investigations of this question will be mentioned.

A comprehensive investigation of steel piles was made in connection with the construction of a 700-foot arch bridge in the southern part of Norway. For the abutments a great number of batter piles with the inclination 4 on 1 were to be driven. The rock, which was found below soft clay at a depth of 70 feet was extremely hard and it was therefore feared that the batter piles would slide on the rock surface and not get a sufficiently good bearing. To investigate this question four piles were driven at a site where the depth to rock was so small that they could be excavated and inspected after driving. Piles No. 1 and 2 were supplied with a normal Oslo-point having a hollow ground bottom. The points of piles No. 3 and 4 were, as an experiment, supplied with a special corona formed with six edges (see Fig. 6). The piles were driven according to the procedure mentioned above.

The results of the inspection of the excavated piles are shown in Fig. 6, illustrating how the pile points appeared after the driving and indicating the movement of the pile points during the last stage of the pile driving. From the figure it is seen directly that pile No. 1 has obtained the most satisfactory bearing on the rock. Almost 100 per cent of the point area was in contact with rock. The second best bearing was observed for pile No. 3. For this pile approximately $\frac{2}{3}$ of the point was in contact with rock. Piles No. 2 and 4 were not sufficiently embedded since the contact area was smaller than 50 per cent.

If these observations are compared with the driving records given in the right-hand part of the figure it is found that pile No. 1 for which the best bearing was obtained also was the pile which was given the largest number of blows (800) after contact. The second best bearing was found below pile No. 3, which received the second largest number of blows (500). The two unsatisfactory piles were only subjected to 130 and 230 blows. The conclusion was therefore drawn that the procedure of driving the point into the rock until a satisfactory bearing of the pile is established, requires a great number of blows. It was, moreover, shown by the experiments that the drop of the ram should be stepwise increased to 25 inches and continued with this fall until no measurable set is observed.

It should be mentioned that the form of the point used for

piles No. 3 and 4, i.e., a corona of six edges, did not prove to be applicable since it did not prevent a sliding of the point on the rock surface. This type of point will, moreover, never obtain a sufficient contact area between steel and rock to keep the stresses from the pile load within reasonable values.

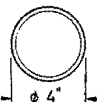
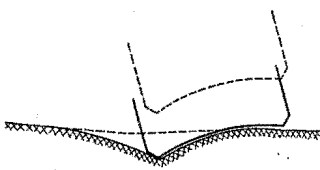
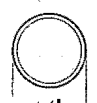
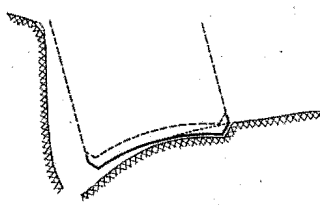
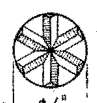
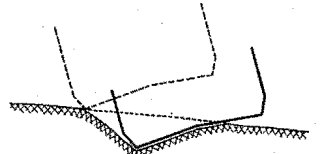
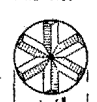
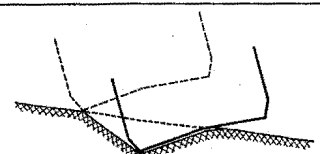
Pile no. Type of point	Bearing on rock	Pile driving data		
		Number of blows	Height of ram, inches	Settlement inches
Pile no. 1  Oslo-point ϕ 4"		100	4-8	0.20
		100	8	0.55
		100	8-10	0.24
		100	8-10	0.12
		100	8-10	0.04
		100	8-10	0.04
		100	28	0.08
		100	28	0
		800		1.27
Pile no. 2  Oslo-point ϕ 4"		100	4-6	3.5
		100	12-14	0.5
		25	24-28	0.08
		5	47	0.04
		230		4.12
Pile no. 3  Corona-point ϕ 4"		100	4-6	0.24
		100	8	0.04
		100	10-12	0.20
		100	16	0.08
		100	24	0.16
		5	47	0.04
		505		0.76
Pile no. 4  Corona-point ϕ 4"		100	4-6	0.04
		100	20	0.20
		25	40	0.08
		5	50	0.08
		230		0.40

FIG. 6. POINT BEARINGS OF FOUR BATTER STEEL PILES DRIVEN TO ROCK AND AFTERWARDS EXCAVATED FOR INSPECTION.

Fig. 7 shows a photograph of the rock below pile No. 1, after the excavation. It illustrates how the point has chiseled a depression in the rock with a concave circular form which gives a direct impression of a satisfactory bearing of the pile. Fig. 8 shows pile No. 1 after the excavation and it can be seen that the point is intact in spite of the

fact that the pile was subjected to 800 blows and that the rock was extremely hard.



FIG. 7.—PHOTOGRAPH OF THE ROCK BELOW POINT NO. 1 IN FIG. 6. The circle indicates the depression caused by the pile point.



FIG. 8.—PHOTOGRAPH OF THE POINT OF PILE NO. 1 IN FIG. 6 AFTER EXCAVATION.

In the described tests the inclination of the pile to the rock surface was 65° - 70° . Some similar tests have, however, been carried out in Gothenburg (Petterson 1941) in which the inclination was 45° . The results of the excavation of the piles after the driving are given

in Fig. 9. In this case the rock was a very hard granite with a smooth surface.

As can be seen from the figure the piles did not bite into the rock immediately after contact. They slid along the surface 4-8 inches before they stopped. With our present experiences this sliding is explained by the relatively large drops of the ram which were used. It

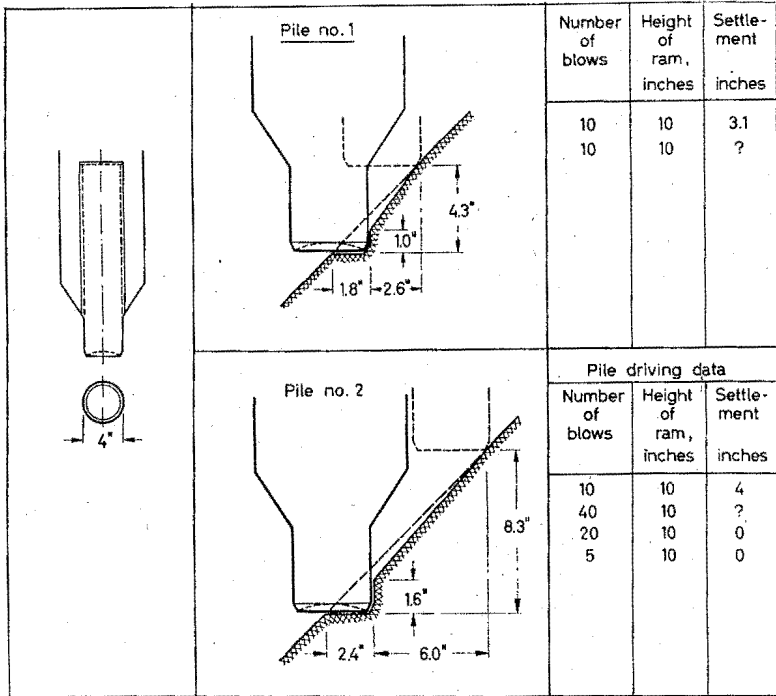


FIG. 9.—TEST DRIVING OF PILES TO HARD ROCK WITH 45° INCLINATION (PETTERSON 1941).

was possible, however, for both piles to establish a toe for the points. In the author's opinion the pile driving was stopped too early. With a sufficient number of blows it would have been possible to continue the chiseling procedure until the total area came into contact with the rock. It is, thus, interesting that pile No. 2, which had the best contact with rock, also is the pile which received the greatest number of blows.

The two described investigations thus show that it is possible

to establish a satisfactory bearing of a pile on rock even if the pile is inclined relative to the rock surface and even if the rock is very hard and has a smooth surface. The results emphasize the importance of supplying the piles with a point of the Oslo-type, and the conclusion can be drawn that the pile driving should be continued with small drops and a great number of blows until a satisfactory toe has been chiselled into the rock.

BUCKLING OF PILES

The danger of buckling of piles in soft clay is a ghost which now and then appears in technical discussions. Hitherto it has been possible each time to waylay the ghost with the aid of one or both of two well known magic spells. The first of these spells is a reference to some theoretical investigations which seem to indicate that there is no danger of buckling even of long piles in soft clay. The second spell is a reference to a loading test of a $\frac{3}{4}$ " \varnothing steel rod which did not buckle even under a load of 3 tons.

The buckling problem is, however, not so simple that it can be dismissed by a reference. There is no doubt that the theoretical considerations, which were developed by Forsell (1918, 1926), Granholm (1929), and Cummings (1938), indicated that the danger of buckling is essentially reduced by the lateral support of surrounding clay. The theories are, however, in general, interpreted in a way that implies that there is no danger of buckling at all and that this problem therefore can be ignored in the design of piles. This interpretation is valid for the majority of piles and, of course, in all cases where the soil is capable of supporting a part of the load by adhesion along the sides of the piles. For the case of steel piles which often are driven down to rock through very soft clay, the danger of buckling has to be considered more carefully.

Considering initially straight piles, the theoretical buckling load is calculated by assuming that the pile will buckle into a sinusoidal curve with a number of half-waves of length l . In a similar way as for free-standing columns a critical load, P_{cr} is found:

$$P_{cr} = \frac{EI \pi^2}{l^2} + \frac{cl^2}{\pi^2}$$

In this formula I is the minimum moment of inertia of the pile and E is the modulus of elasticity of the pile material.

The coefficient of lateral displacement, c , depends on the size and the shape of the loaded area and on the modulus of compressibility of the clay¹. It should be noticed that c is equal to the horizontal subgrade reaction (Terzaghi 1955) multiplied by the width of the pile. For Norwegian clays the modulus of compressibility will vary between 150 and 700 lb/in². For typical values of length and width of steel piles, c will vary from 100 to 600 lb/in².

In the above expression, l is that value which gives a minimum P . This value, which is called the critical length of the pile, is

$$l_0 = \pi \sqrt[4]{\frac{EI}{c}}$$

Introducing this value in the expression for P_{cr} , one obtains:

$$P_{cr} = 2 \sqrt{c EI}$$

This expression permits a calculation of the buckling load of a straight pile if c is known. Based on this formula it is possible to introduce a criterion to decide whether or not buckling should be considered in the design of straight piles. This depends, of course, on whether the buckling load is higher or smaller than the load which results in yield stresses in the pile material. If, therefore,

$$P_{cr} \geq \sigma_{max} A$$

where σ_{max} is the yield stress of pile material
and A is the cross-sectional area of pile,

it can be concluded that the danger of buckling of straight piles can be ignored. By introducing the buckling load into this expression, it is found that buckling needs to be considered only if

$$\frac{I}{A^2} \leq \frac{\sigma^2_{max}}{4 c E}$$

In any particular case the right side of this equation will be fairly constant since it depends upon the pile material and the properties of the soil. Values of I/A^2 for typical pile sections are given in Table I. It is seen that I/A^2 depends only upon the shape of the section, and its value is smallest for piles with a compact cross section.

¹The use of the coefficient of lateral displacement implies the assumption of proportionality between unit load and deflection. For the considered case of slender piles this assumption has been proved to be valid by Biot (1937).

For the case of steel piles with $\sigma_{\max} = 52000 \text{ lb/in}^2$ and $E = 3 \times 10^7 \text{ lb/in}^2$ the use of a very low value of c , e.g. 75 lb/in^2 , it is found that there is no danger of buckling of initially straight piles as long as

$$I/A^2 \geq 0.30$$

Comparing the values of I/A^2 in Table I with the critical value 0.30 for steel piles in soft clay, it can be seen that there is no danger

TABLE I—VALUES OF I/A^2 FOR DIFFERENT PILE PROFILES.

Profile	Description	I/A^2
●	Round steel bars	0.080
■	Square steel bars	0.083
⌚	Train rails	0.17
⌚	H-sections 8"×8"	0.31
⌚	H-sections 10"×10"	0.34
○	Steel tubes $\phi 12'' \delta = 0.8''$	0.36
□	Steel box sections of welded angles 8"×8"	0.47

of buckling for the H-sections (except the smallest x-section), for the tubes or for the section with welded angles. The steel rods and the rails show, however, such a low value of I/A^2 that there is an actual danger of buckling if they are used as piles in very soft clay, and their bearing capacity will, therefore, be governed by considerations of the buckling load.

In the above calculations it was assumed that the pile was initially straight. Now, it is impossible to drive all piles perfectly straight. As a misalignment obviously will increase the danger of

buckling, it is of interest to develop a theory to include piles with initial imperfections. An attempt at this was made by Glick (1948) and in a later investigation Gibson (1954) improved the theory by including the danger of overstressing the soil which surrounds the pile. In both of these theories, however, an over-simplification of the theory has been made, since the effect of the soil reaction on the bending moment has been ignored. In 1951 Walter presented a solution which is satisfactory because it includes this effect. There is, however, another factor which Walter does not consider and which has an appreciable effect on the danger of buckling. This factor is the initial stresses which exist in a pile because of the curvature imparted during driving. There are, thus, no available theories which are complete enough to permit an evaluation of the danger of the buckling of piles in practice, and, moreover, full scale tests are needed to investigate the validity of the assumptions on which the theories are based.

From the theoretical investigations it can, however, be concluded that the failure loads of piles are sensitive to initial imperfections in the piles. It is, therefore, in practice a necessary requirement that the maximum eccentricity of a pile must be below certain limits in order not to reduce the bearing capacity. In this connection it can be mentioned that since 1935 the building authorities in Oslo have rejected curved piles which showed a radius of curvature smaller than 1200 feet. The available theories support this way of expressing the requirement regarding the alignment of a pile, and for typical pile dimensions they also indicate that the Oslo rule of a minimum curvature of 1200 feet is a reasonable but slightly conservative specification.

In order to enable a control of the alignment of a pile, the building authorities in Oslo require that a $1\frac{1}{2}$ " or 2" tube be welded to the pile before it is driven, and that the alignment of the pile be controlled by lowering an inclinometer into the tube for measuring the inclination of the pile at different depths. The Norwegian Geotechnical Institute has recently built an inclinometer, of outside diameter only $1\frac{1}{4}$ inch, of a type developed by Plantema (1953). It uses strain gauges and gives at each point the inclination in two planes perpendicular to each other. Fig. 10 shows the result of a measurement of the alignment of an H-pile driven for a bridge in Oslo.

The explanation why the danger of buckling has been discussed

in some detail in this paper is that recently in Norway a few steel piles buckled during loading tests. This is, as far as is known to the author, the first time buckling of piles has been established; therefore, it is of interest to study these piles. The piles were driven

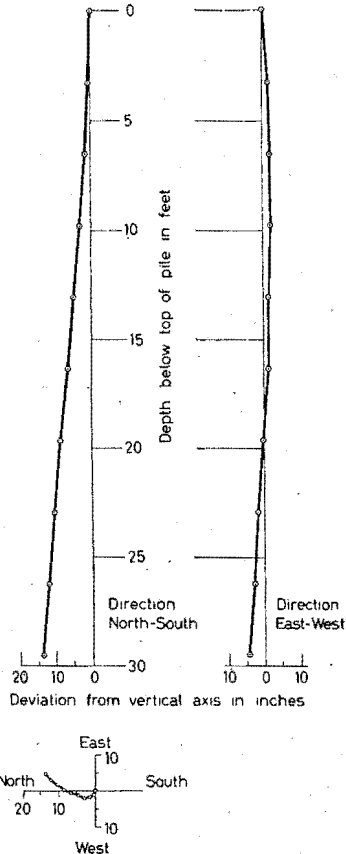


FIG. 10.—RESULT OF MEASUREMENT OF THE CURVATURE OF A STEEL PILE.

for underpinning a church in Trondheim in Norway. Professor A. Brandtzæg at the Technical University in Trondheim and Mr. E. Harboe, consulting engineer, Trondheim, have written a paper about the work which will be published in English in the not too distant future. With permission of the Authors the loading tests will be briefly mentioned here.

Before selecting the pile type for the underpinning, loading tests were carried out on five steel piles driven to rock. Four of the piles were 1¼" steel reinforcing rods. The fifth pile was a train rail. During the loading tests all the piles failed. The four steel rods were pulled and it was observed that they had buckled approximately at the same depth.

For the underpinning it was decided to use normal railway rails with a weight of approximately 23 pounds per foot and the piles were

TABLE II—DATA OF STEEL PILES LOADED TO FAILURE AT LADEMOEN, TRONDHEIM (BRANDTZAEG AND HARBOE, 1957).

Pile no.	Pile type A = area of cross section I = moment of inertia	Pile length		Loading tests				Esti- mated k lb/in ³	Calculated	
		to rock feet	in clay feet	P _{max} tons	σ _{max} lb/in ²	δ _{max} in	Buckling length		critical buckling length	buckling load (initially straight pile) tons
2	• ϕ 1¼ in	118	79	9	18 300	c 0.9	4' 3"	110	3' 4"	16.3
3	A = 1.1 in ²	121	79	9	18 300		6' 6"	110	3' 4"	16.3
4	I = 0.096 in ⁴	89 (?)	79	16	32 600		3' 1"	110	3' 4"	16.3
5		121	79	11	22 400		5' 3"	110	3' 4"	16.3
6	I h = (?) A = 6.8 in ² I _{min} = 8.4 in ⁴	c 118	79	150	49 400	2.9	?	110	10' 1"	153
7	I h = 4.5 in b = 4.7 in A = 8.7 in ² I _{min} = 4.8 in ⁴	118	98	120	40 000	4.7	14' 1"	110	8' 9"	116

straightened before driving. A great number of the piles were test loaded after driving. Some irregularities occurred during the tests on one of the piles since unusually large settlements were observed. The loading test was therefore repeated. The load was increased up to 120 tons and kept constant at this value. After 2½ hours it was observed that the load had dropped down to 40 tons and it proved impossible to increase it above this value. It was decided to pull the pile and it was thereby found that the pile had buckled over a length of approximately 14 feet, again at the same depth as the five test piles. The data of the piles and the results of the loading tests are given in Table II.

Borings carried out at the site have shown that under an upper layer of gravel with a thickness of 5 to 10 feet there is a deep layer of normally consolidated soft clay. The water content of the clay is

well above the liquid limit, so the clay is very sensitive. The shear strength of the clay varies from 2.0 to 2.8 lb/in². At a depth of approximately 100 feet there is a layer of sandy stiff clay with stones. The depth to rock is approximately 120 feet.

A comparison between soil conditions and observations of the buckling length of the five piles described above showed that the failure of the piles took place in the very upper layers of the soft clay. From these clay layers undisturbed samples were taken for investigation in the laboratory and, of course, it was of special interest to find out the modulus of compressibility which controls the danger of buckling of piles. Based on an interpretation of the stress-strain curves found by compression tests the value of c , the coefficient of lateral displacement, was found to vary between 100 and 130 lb/in² for the considered piles.

As mentioned above the danger of buckling of a pile depends to a high degree on the initial eccentricity. It is therefore difficult to interpret the test results in Table II because the initial form of the piles was not investigated. Considering the four steel rods equal in diameter and steel quality and driven within small distances of each other, the failure loads, however, vary from 9 to 16 tons. This difference in failure loads can only be explained by different initial shapes after the driving and it is thus obvious to assume that steel rod number 2, which carried the highest load, had less eccentricity than the three other rods. If it is assumed that rod number 2 was initially straight, it is possible to calculate very simply a theoretical failure load. Using c equal 115 lb/in² as found on the average by the laboratory tests, a calculation shows that the critical load of the rod should be 16.3 tons which is in surprising agreement with the observed 16.0 tons. The theoretical critical length of the pile is 3.3 feet and also this value agrees with the observed length of buckling, 3.1 feet.

If, in the same way, it is assumed that pile number 5, the train rail, was initially straight, a theoretical failure load of 154 tons can be calculated, compared with the observed 150 tons. Pile number 6, the rail, gives under the same assumption of initial straightness a failure load of 119 tons, whereas the loading test resulted in a failure load of 120 tons. Thus, the agreement between loading tests and theoretical calculations are surprisingly good also for these two piles.²

²The buckling length of pile number 6 was observed to be 14 feet. The theoretical critical length is, however, only 9 feet.

The loading tests from Trondheim are thus interesting to study. In the first place they prove that buckling of a pile actually can occur, and has to be considered in soft clays. In fact, the allowable loads on the piles for the underpinning of the church were reduced in order to obtain a satisfactory safety factor against buckling of the piles. In the second place the good agreement between observed and calculated failure loads indicate that the theoretical approach works satisfactorily for initially straight piles. Finally, the observations prove that the danger of buckling increases considerably if the piles are not initially straight and they, thus, demonstrate the necessity of keeping a certain control on the initial shape of piles in soft clay.

CORROSION OF STEEL PILES IN CLAY

In connection with the use of steel piles, there is a question which requires an especially careful consideration, that is, the danger of corrosion.

When the Building Authorities of Oslo in 1935 accepted the use of unprotected steel piles, the question of corrosion was seriously discussed and in this connection a number of old steel piles were pulled for inspection. The conclusion was that steel piles were permitted in clay, provided that the pH-value of the pore water of the clay was higher than 7.0 and that there was no movement of the ground water.

Today our knowledge of corrosion has increased considerably and we know that corrosion is a complicated electro-chemical process. During corrosion, the iron atoms are converted into positively charged iron ions with a corresponding number of negative electrons. The condition of continuous corrosion is that the negative electrons are consumed or removed, a process which is called cathodic depolarization.

If steel is in contact with acid, the negative electrons are consumed by the protons of the acid, and a continuous corrosion is observed. This process is controlled by the concentration of protons, i.e., the pH-value. This fact may explain the criterion used by the Oslo Building Authorities. There are, however, several other possibilities of removing the electrons and this criterion is then not generally valid.

It is a well known fact that steel corrodes rapidly if in contact with oxygen under moist conditions—for instance near sea water.

Similar processes take place in marine clays in which oxygen is present chemically bonded in highly oxidized compounds. There is thus no reason to believe that the danger of corrosion can be neglected even in clays with low permeability. The corrosion will depend to a higher degree on the chemical compounds of the clay than on the permeability.

A continuous transport of the electrical charges requires a certain degree of conductivity of the surrounding clay. The conductivity is, therefore, an important factor for the corrosion. A high conductivity will thus increase the danger of corrosion, provided the necessary chemical conditions are present, but it will not itself be a sufficient criterion for expressing the danger of corrosion.

The danger of corrosion of steel piles in clay is thus governed by two factors: the cathodic depolarization and the conductivity. In a homogeneous clay it is supposed that the ratio of the degree of depolarization to the logarithm of the electric resistivity is a figure which directly expresses the danger of corrosion. The degree of depolarization is understood to be the relative value compared with a maximum value observed for instance in salt water saturated with oxygen.

Any inhomogeneity in the soil along the pile will promote and concentrate the attack on the pile. With the relatively low soil temperature in Norway, the effect of sulphate reducing bacteria is believed to be small. In countries with warmer climates this may, however, be an important factor in the process of corrosion.

In order to obtain reliable measurements of the factors controlling corrosion, Dr. I. Th. Rosenqvist at the Norwegian Geotechnical Institute has constructed a sounding device with which the depolarization and the resistivity are measured directly "in situ". The corrosion penetration-device consists of a $1\frac{1}{4}$ " steel tube supplied with a magnesium point insulated from the tube as shown in Fig. 11. The sounding device is driven into the clay and at various depths two types of measurements are made.

In the first place the electric resistivity of the clay is measured by applying a current between point and tube. By using an alternating current with 1000 cycles per second, no depolarization takes place.

The next step is to measure the galvanic current passing from the tube to the magnesium point when they are short circuited. This current indicates how fast the negative electrons are removed from the

surface of the steel tube, and it will, therefore, give an expression for the cathodic depolarization of the clay. If the measured current of the short circuited system is multiplied by its electric resistivity, the "electromotive force" is obtained. Preferably this is expressed by the degree of depolarization, obtained by dividing it by the maximum value, for instance measured in salt water saturated with oxygen.

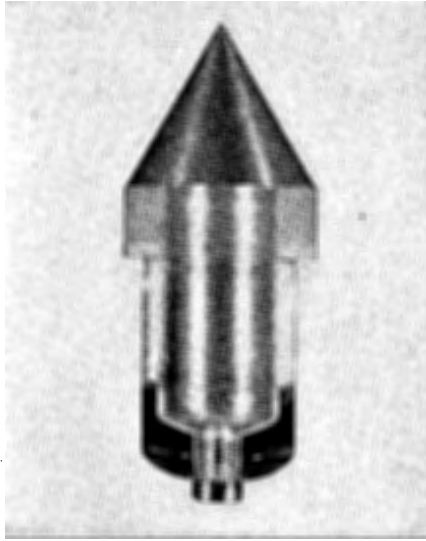


FIG. 11.—PHOTOGRAPH OF THE CORROSION POINT FOR IN SITU MEASUREMENTS OF CONDUCTIVITY AND DEPOLARIZATION OF SOIL.

In Table III it is shown how the values obtained by the corrosion device are interpreted. Depending on the degree of depolarization and the electric resistivity measured by the corrosion device, the corrosivity is described by a classification using five different groups. The intermediate group is just at the dangerous limit for steel piles since it represents an average corrosivity of 1/1000 of an inch per year.

The calibration of the above described procedure was made by measuring the corrosivity on sites where old steel piles could be pulled for inspection.

Fig. 12 shows three examples of measurements of the corrosivity from three different sites in Norway. In each profile the measured

values of the degree of depolarization and the logarithm of the resistivity are plotted against the depth. The interpretation is given as the group number in the classification system shown in Table III. The figures show moreover the results of an inspection of old steel

TABLE III—CLASSIFICATION OF THE DANGER OF CORROSION OF A SOIL BASED ON ITS CONDUCTIVITY AND DEGREE OF DEPOLARIZATION.

		<i>Degree of depolarization in per-cent</i>			
		<i>0-40</i>	<i>40-60</i>	<i>60-80</i>	<i>80-100</i>
<i>Electric specific resistivity of clay in ohm centimeter</i>	<i>0-50</i>	2	3-4	4	5
	<i>50-250</i>	2	3	4	5
	<i>250-1250</i>	2	2-3	3	4
	<i>1250-6250</i>	1	2	3	3-4
	<i>6250-</i>	1	1	2	2-3

Classification of corrosivity:

- Group: 1 Very low corrosivity*
2 Low corrosivity
3 Intermediate corrosivity
(app. 1/1000 inch pr. year)
4 High corrosivity
5 Very high corrosivity

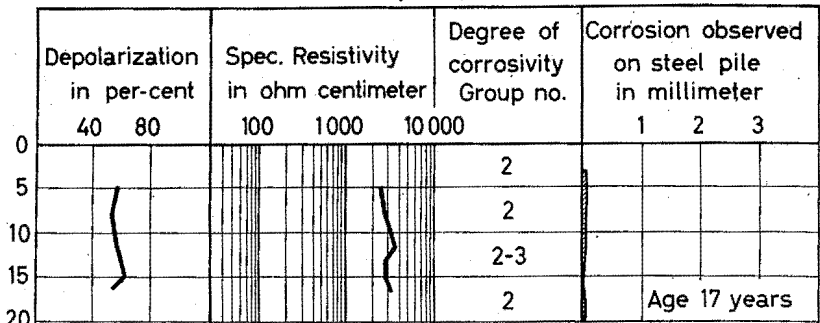
piles which were pulled. The measured attack is directly plotted in millimeters at the different depths.

The example from Hafslund in Sarpborg is a silty clay with an intermediate degree of depolarization and a relatively high resistivity. According to the values obtained by the corrosion test the clay is classified as having a low corrosivity. This is confirmed by observations from a pulled steel pile which was driven 17 years before the inspection. The steel pile showed an attack smaller than 3/1000 of

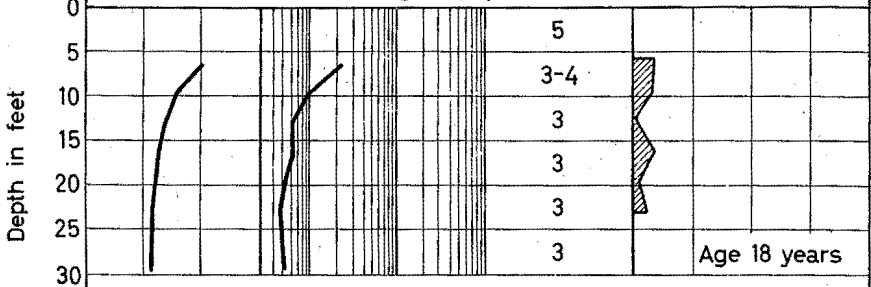
an inch which corresponds to an average of 1/10,000 of an inch per year.

The next example is a typical Oslo clay of marine origin. Due to the salt content the resistivity is as low as 50 OhmCm. The

Hafslund, Sarpsborg



Oslogaten, Oslo



Persgangen, Oslo

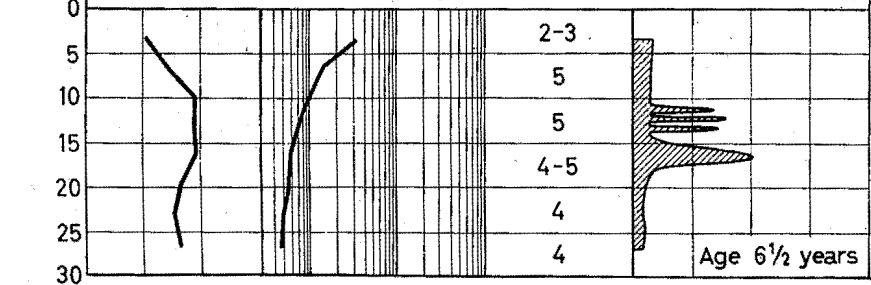


FIG. 12.—EXAMPLES ON RESULTS OF IN SITU MEASUREMENTS WITH THE CORROSION POINT AT THREE DIFFERENT SITES, WHERE ACTUAL CORROSION WAS OBSERVED ON STEEL PILES.

depolarization is, however, low to intermediate with a marked increase in the upper crust. In spite of the low resistivity the clay shows an intermediate corrosivity and this was also confirmed from an 18-year-old steel pile which was pulled for inspection. The corrosion varied from 1/100 to 2/100 of an inch, this means 1/2000 of an inch per year.

The third example in Fig. 12, Persgangen, is also a clay from Oslo with the typical low resistivity of the marine clays. In this case, however, the depolarization was as high as 77 per cent. The conditions are very unfavorable in this profile and a 6½-year-old steel pile which was pulled, showed serious corrosion up to 1/10 of an inch. This corresponds to an average corrosion of more than 1/100 of an inch per year.

Based on a great number of measurements of the corrosivity of Norwegian clays it has been proved that the danger of corrosivity of steel piles is in general low in spite of the salt content of the marine clays. In some clay layers a high degree of depolarization was found—for instance in Persgangen in Fig. 12—and the corrosivity may then be very dangerous even if the clay has a low permeability.

The Building Authorities of Oslo now require that steel piles can be used for foundations only if an investigation with the corrosion point has proved that there is no danger of corrosion.

In cases where the corrosivity is considered too high for normal use of steel piles, this type of pile may be used if provided with cathodic protection. Today, such protection is provided at seven buildings in the Oslo area. Normally, a low voltage direct current and inactive graphite anodes are used. The maximum current density on the anodes has been 10 amp. per square meter surface. The potential of the piles is normally depressed to -880 millivolts towards a copper/copper sulphate half-cell. In one case galvanic protection by means of cast magnesium anodes is installed. In case of clays of low permeability and with low hydraulic gradients, such a protection may be active a long time after the magnesium has been consumed.

LOADING TESTS ON STEEL PILES TO ROCK

The most reliable procedure for predicting the behaviour of a pile foundation is to carry out loading tests on single piles and to study the results. In this connection it is useful to compare the results with available results of other loading tests. The results of a

number of loading tests on piles to rock are collected in Table IV. The majority of the tests are on steel piles, but some tests on con-

TABLE IV—LOADING TESTS ON PILES TO ROCK.

Pile No.	Location	Pile Type and Dimensions			Ground Conditions		Driving Acceleration after Contact	Loading Test					Remarks	
		Cross-Section	Point	Length feet	Soil	Rock		Loading Programme	Duration	Pen. Rate	Rate of Pen.	Pen. per Foot		
1	Middeltrom 1933 (Berzell 1938)	Reinforced Concrete 45" dia	Steel Point	45	Sandy Clay	Hard, horizontal	0-30-0	30 hrs for 24 hours	30	570	0.0'	0.0'		
2	Shannon (Berkeley 1930) (Don Dixon)	• 30" A=0.44 m ²	No Point	20	Clay	Granite	0-3-0	3 hrs for one month	3	14 900	0.3'	6	0.1'	
3	Sarregau, Ode 1930 (F. Gerner)	9 1/2" x 9 1/2" x 11 1/2" Section A=11.2 m ²	No Point	101	Clay	Horizontal	4 000 lb Ram 6'-6" Fall Height	0-700-0	300 hrs	160	77 000	1.7'	1.2	0.5'
4				103					20	77 000	1.6'	1.6	0.5'	
5				105					210	77 000	1.5'	1.4	0.5'	
6		12 1/4" x 12 1/4" x 12 1/4" Section A=17.4 m ²		105		3'-3" Fall Height			70	76 300	1.4'	1.4	0.5'	
7				105					700	76 300	1.1'	1.0	0	
8				105					700	24 800	1.6'	1.6	0.5'	
9	Falkenberg, Ode 1931 (Garnstedt 1934)	4 1/2" x 4 1/2" x 17 7/8" A=16.9 m ²	Steel Point 4" dia	482	Clay	Hard, slight inclination			181	23 500	0.8'	1.0	0	
10	Falkenberg 1930 (Hafstrom 1931)	6 1/2" x 6 1/2" x 18" Section A=15.7 m ²	Steel Point 3"	55	Clay	Hard granite	7 1/2 ton Ram 5'-0" after Contact		Sheet Duration	100	24 400	0.5'	0.9	0
11				98			5'-0"		97	23 700	1.0'	0.0	0	
12				154		45° inclined	6'-0"		100	27 000	1.0'	0.9	0	
13		8 1/2" x 8 1/2" x 18" Section A=16.1 m ²	Steel Point 3"	60		Hard granite, horizontal	4 1/2 ton Contact 200 blows 5 lbs = 1'-0" 5 = 3'-1"	0-100-0 -100-0	100 hrs for 2 hours	100	45 600	0.5'	0.9	0
14				65		65° inclined	After Contact 14 tons 2 hrs = 1'-0", 5 = 3'-0"	0-100 4 hrs = -100-0	150 hrs for 0.5 hours	170	18 500	1.2'	1.0	0.5'
15	Franskrug, Ode 1940 (F. Seiner)	Reinforced Concrete 45" dia	Steel Point 4" dia	85	Clay	Hard	0-167-0	157 hrs for 17 months	167	2 500 approx	1.7'	1.5	0.1'	
16	Rubin 1946 (Cassagrande 1947)	14 1/2" x 7 1/8" Section A=77.7 m ²	No Point	144	Clay	Soft horizontal	5 = 0.1 m/10 sec	0-100-0 -100-0	24 hrs for 30 hrs for 5 hrs	100	19 000	0.5'	0.8	0.1'
17	Stromstad 1940 (Borchsenius 1953)	8" x 8" x 14.5" Section A=12.6 m ²	Steel Point 4" dia	13		Ham shales 1.35' thick 1.60'	5 = 1' after Contact 5' = 1' 0" approx 5' = 0' approx	0-40-0-0	90 hrs for 4 mos	90	15 600		6.2'	Failure of 45 lbs
18				16					90	15 600		5.0'		
19	Horsingryg 1953 (Ebersten 1954)	4 1/2" x 4 1/2" A=7.73 m ²	Steel Point 4" dia	85	Clay Gravel	Gravelly slight inclination	No approximate Acceleration after Contact	Repeated loading and unloading	80	17 700	0.5'	1.0	0	
20		8 1/2" x 8 1/2" Section A=16.1 m ²		85					120	18 400	0.5'	1.0	0	
21	Oslo 1954 (Gronhaugen)	Reinforced Concrete 45" dia	Steel Point 4 1/2"	83	Clay	Horizontal	After Contact 250 blows with 4 1/2 ton = 1'-0" 5 = 2.05' 7 1/2 ton	Repeated loading and unloading	Sheet Duration	134	1 940 approx	0.1'	0.1'	0
22	Isakmen Church Foundation 1956 (A. Brandrup 1955)	4 1/2" x 4 1/2" A=7.73 m ²	None	105	Gravel Clay Gravel	Relatively horizontal	2 700 lb Ram	9-105-0-100-0	Sheet Duration	119	35 700	1.6'	0.8	0.1'
23				105-171					120	35 700			0.1'	
24				115					4	30 700				
25	Frederstad 1955	8" x 8" x 13.8" Section A=10.2 m ²	Steel Point 4" dia	85	Clay	Hard granite, horizontal	5 000 lb Ram Control Driving, Approx 1000 blows after Contact	0-700-0-100-0	100 hrs for 160 hours	260	22 700	0.8'	1.0	0
26				69		Slight inclination	5-1' after Contact approx 1000 blows	0-700-0	260	22 700	0.1'	1.1	0.1'	
27				71			5-1' after Contact	0-700-0	200	22 700	0.5'	0.9	0	
28				65			5-0' after Contact	0-174-0	174	20 000	0.0'	1.1	0.1'	
29	New York 1939 (Stek 1948)	14 1/2" x 8 1/8" Section A=70.7 m ²		188	40% wet clay and 20% sand 100% sand Sand 53.8' from ground		14 000 lb Ram 2' 5" Fall Height Final Acceleration = 0.6724v	0-700-0	100	15 000	0.0'	0.8	0.2'	
30	New York 1947 (Stek 1948)	12 1/2" x 5 3/8" Section A=15.6 m ²		70		Soft and fine sand	0-500 lb Ram 3' 0" Fall Height Final Acceleration = 0.757024v	0-150-0	150	15 000	0.60'	1.0	0.16'	
31		12 1/2" x 7 1/8" Section A=19.5 m ²		80					150	15 000	0.35'	0.7	0.05'	
32		14 1/2" x 8 1/8" Section A=70.7 m ²		107					180	17 800	0.45'	0.8	0.07'	

crete piles driven to rock have also been included. A few of the tests will be discussed in detail below:

Sprenga, Oslo (pile numbers 3-8)

In connection with the construction of a coal bunker in the harbour of Oslo in 1930, the first loading tests on steel piles were carried

out in Norway.³ The piles were H-piles, but unlike the later piles they were not constructed with a special point.

If the results of the six loading tests are compared with the results of the other piles in Table IV, it is seen that they are distinguished by their larger permanent settlements. Whereas the permanent settlements of the successful piles in Table IV vary from 0 to a maximum of a quarter of an inch, the six piles at S renga show two to three times these values.

It is very likely that the permanent settlements of the six piles are due to the fact that they were driven without a special point. The driving of the piles was stopped when rock was reached and the contact between the end of the steel pile and the rock has, therefore, probably been limited to a small area at one of the flanges. If this has happened, the 210-ton-load would result in stresses at contact which by far exceed the yield point of the steel and the pile would deform until a contact area was established which was sufficiently large to transmit the load to rock.

Warehouse Gothenburg (pile numbers 10-14)

In connection with the first application of steel piles in Sweden in 1940, a comprehensive series of loading tests was carried out (Petterson 1941). The piles were H-piles provided with a normal Oslo-point.

Three piles, numbers 10, 11 and 12 in Table IV were initially driven as test piles. They were test loaded and afterwards pulled for inspection. Under two of three piles, numbers 10 and 11, the rock was only slightly inclined. The piles obtained a good bearing in the rock and under the test load the piles showed only small settlements of purely elastic character. By inspection, the piles were shown to be intact.

The third pile, number 12, was driven at a place where the surface of the hard granite rock showed an inclination of approximately 45°. During the driving, the point penetrated more than six feet after contact before a firm bearing with the rock was established. Also this pile experienced only small and elastic settlements in the loading test. By pulling the pile a sharp bend four to five feet above the point was observed.

It is the opinion of the author—contrary to the approval of

³Private communication of Mr. F. Selmer.

the test results by the harbour authorities—that a pile like this should not be accepted as a part of the final foundation. The promising results of the loading tests may be due to the fact that the pile is very long—154 feet—and under the short term loading, the skin friction from the surrounding clay concealed the imperfections of the lowest part of the pile. If the load had been kept constant over a

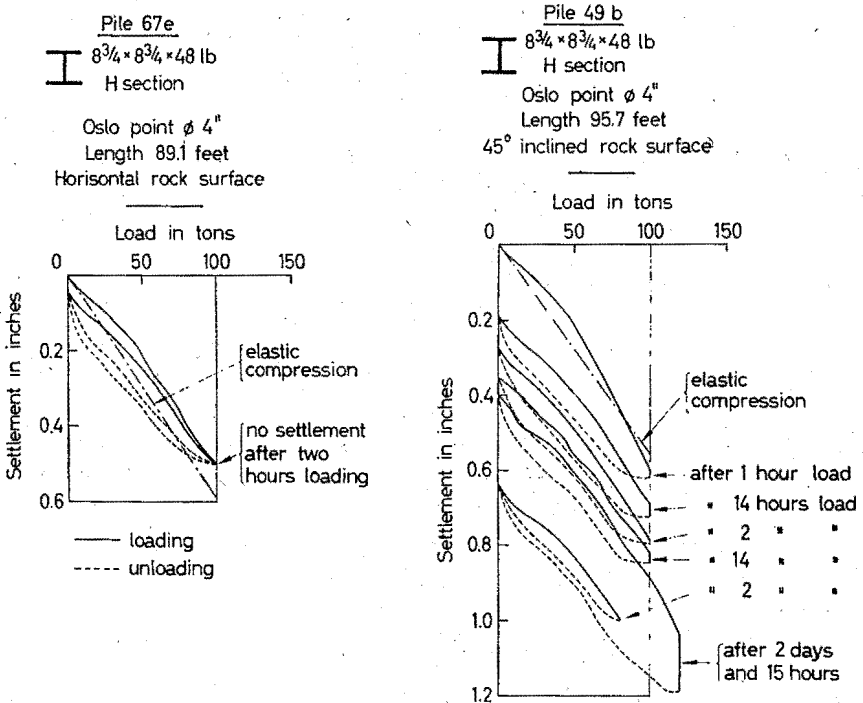


FIG. 13.—RESULTS OF LOADING TESTS WITH TWO STEEL PILES IN GOTHENBURG (PETTERSON 1941).

longer period or if the pile had been unloaded and reloaded several times, the settlements would probably have increased considerably.

The justification of a rejection of pile number 12 is supported by the loading tests on the two piles, numbers 13 and 14, at the same site. Pile number 13 was driven at a place with a horizontal rock surface and it obtained a perfect bearing. The results of the loading test are shown at the left-hand side in Fig. 13 and are very satisfactory since the settlements are small even for a repeated loading. Pile

number 14 was driven to a 45° inclined rock surface and the point did not get a bearing in the rock before a further penetration of three feet. The results of the loading test are shown at the right-hand side in Fig. 13 and it will be noticed that the settlements during the first loading are small, approximately corresponding to the elastic compression of the pile. A repeated loading and unloading, however, and a constant load over longer periods proved to result in increased permanent settlements. The test loading of the pile ended with a permanent penetration of the pile of 0.6 inches, which is large compared with the piles which obtained a satisfactory bearing in rock. It is very likely that pile number 12 in Table IV would have shown a similar behaviour, if the loading tests had been carried out in a similar way.

It should be mentioned that pile number 14 is considered unsuccessful, not because the absolute settlements, 0.6 inch, are considered dangerous, but because the pile in the final structure has to co-operate with successful piles like pile number 13. This, of course, would result in an unequal and unforeseen distribution of the loads.

Concrete pile, Oslo (pile number 15)

A concrete pile provided with an Oslo-point was driven to rock in 1946 and test loaded over a period of 17 months.⁴ Since the load has been kept on the pile for a longer time than in any other test, the direct results of the settlement observations are shown in Fig. 14. The increase in settlements which were observed during the 17 months amounted to $\frac{3}{4}$ inches, of which the major part occurred during the first few months. The final settlements were 50% larger than the elastic compression of the pile. This may be explained by creep of the concrete and the rock below the steel point or it may be a result of an additional loading of the pile by negative skin friction during reconsolidation of the surrounding clay. It is worth noticing that the permanent set of the pile was less than a quarter of an inch.

Warehouse Slemmestad (pile numbers 17 and 18)

The only loading tests during which piles failed due to a penetration of the point into the rock are numbers 17 and 18, carried out in Slemmestad in 1944 (Berntsen 1955). The rock was an alum shale, a rather soft and fissured rock. The inclination of the rock surface

⁴Private communication of Mr. F. Selmer.

varied between 35° and 60°. The piles were H-profiles and were provided with a 4-inch diameter Oslo-point.

A number of piles were test loaded and because they failed at a load as low as 30 to 60 tons, it was decided to excavate two piles

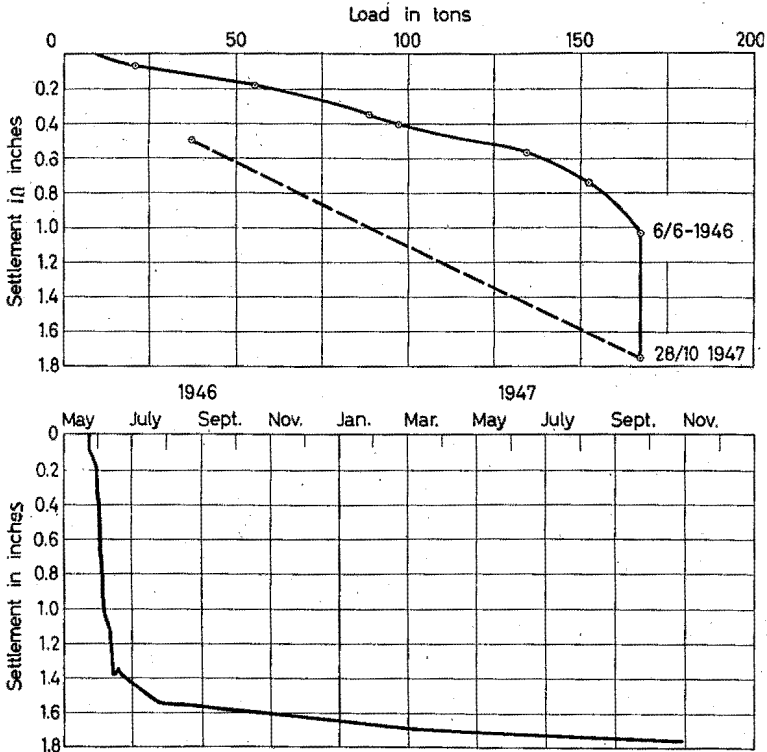


FIG. 14.—RESULTS OF 17 MONTHS LOADING OF A 87 FEET LONG CONCRETE PILE IN OSLO. THE PILE, WHICH WAS SUPPLIED WITH A 4" DIAMETER OSLO-POINT, WAS DRIVEN APRIL 23-24, 1946.

which had a length of only 15 to 20 feet. The result of the inspection of the piles is shown in Fig. 15. It proved that even if the points had slid on the rock surface, they finally obtained a hold in the rock and a bearing was established after 2-4 inches of penetration into the rock. During the loading test of pile number 18, the point of the pile penetrated 6 inches further into the rock, squeezing the rock so that a crater was formed around the point.

Pile number 17 was test loaded after the excavation in order to observe what happened with the piles during a loading test. At 40 tons load the pile started to settle and the point was directly observed to penetrate into the rock. As the point came deeper below the rock surface, a greater load was required to bring the point to failure, and after 6 inches settlement the bearing capacity had increased from 40 tons to 90 tons. The explanation of the small failure loads of the

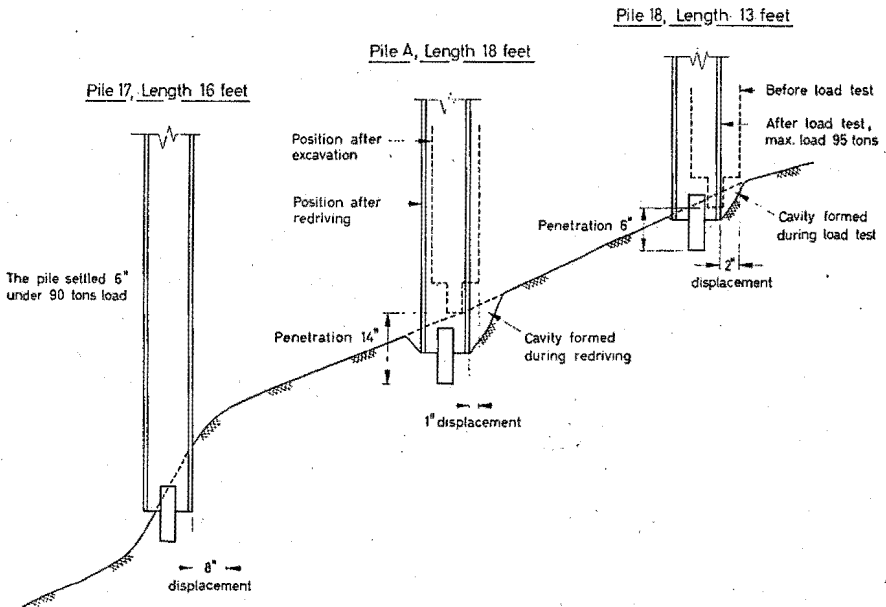


FIG. 15.—STEEL PILES IN SLEMMESTAD EXCAVATED AFTER THE DRIVING (BERNTSEN 1955).

piles was, thus, that the bearing capacity of the alum shale was exceeded. The 30-40 tons failure loads of the piles correspond to a unit pressure at the rock of 4000-7000 lb/in². This value represents, thus, the bearing capacity for a circular point at a depth of one to two inches. At a depth of 6-8 inches, the bearing capacity increased to double the value.

This example demonstrates the importance of considering the strength of the rock for pile foundations. If the rock is poor with low bearing capacity, the piles should either be designed for a smaller load or the diameter of the point should be increased. Another solu-

tion is to drive the piles so far below the rock surface that the bearing capacity has increased sufficiently, a remedy which may require longer pile points than the usual Oslo type.

Lademoen Church, Trondheim (pile numbers 22-24)

In connection with the underpinning of a church in Trondheim, a great number of piles were test loaded. A few of them buckled and they have been discussed previously. The majority of the piles, however, behaved satisfactorily during the testing. Representative data for this group of piles are given in Table IV, piles numbers 22 and 23.

One pile, number 24, failed during the loading test without buckling. The explanation of the small bearing capacity of this pile is probably that the pile was not driven sufficiently hard to ensure the contact between point and rock. The pile was actually a replacement pile since the first pile bent during the pile driving. This fact may explain why the replacement pile was driven with extreme care and the pile driving was stopped as soon as the rock was supposedly reached. This means that the pile was not secured to the rock and the bearing capacity was, therefore, low. That this explanation is true, was confirmed by the observation that the bearing capacity of the pile increased considerably as the pile penetrated a further 4-5 inches.

Bridge Abutment Fredrikstad (pile numbers 25-28)

Of special interest are the loading tests on the steel piles driven for the abutments of a 650-foot arch bridge at Fredrikstad. The piles are of the hollow box type, made by welding two angle profiles together. They are provided with a standard Oslo-point of hardened steel.

The rock at this site is an extremely hard granite, in fact, it is the hardest rock met with in Norway. The pile driving after contact with rock, was made according to the rules described above with step-wise increase in the fall of the hammer and each pile was secured to the rock by more than 1000 blows. Finally, all piles were tested by deformation measurement.

Pile number 25 was loaded with 200 tons—which is two times the design load—for a period of a month. The results of the settlement observations are shown at Fig. 16, and, as seen from the figure, the immediate settlement under the 200 tons loading was approxi-

mately $\frac{3}{4}$ inches and the additional settlement during the one month loading amounted to only 0.2 inches. The permanent set of the pile after removing the load was negligible. The total maximum settlement after one month's loading is approximately equal to the elastic compression of the pile, which must be considered as a very satisfactory result.

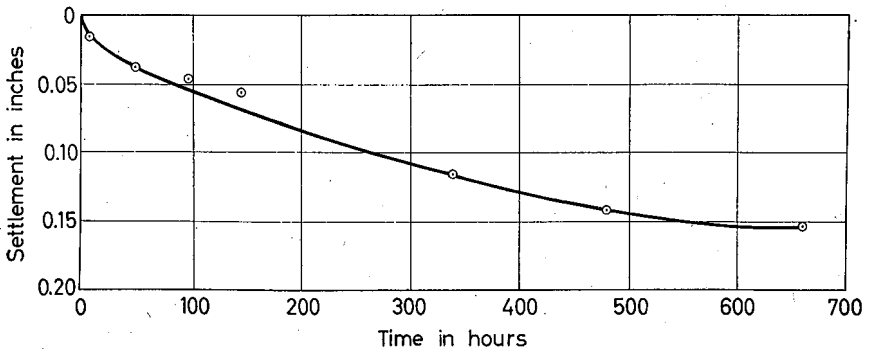
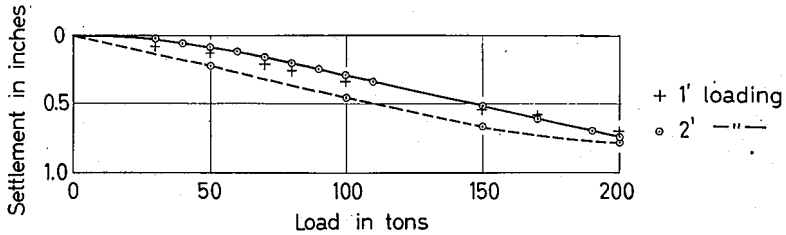


FIG. 16.—LOADING TEST ON A 85 FOOT LONG STEEL PILE TO ROCK AT THE BRIDGE AT FREDRIKSTAD.

Pile number 26, which was only loaded over a period of three days, showed similar properties, but the permanent set was 0.2 inches. The straightness of the pile was investigated before and after the test and whereas the pile was straight before the loading test, it showed a slight curvature after the testing.

The piles numbers 27 and 28 were batter piles with an inclination of 4 on 1. The purpose of the loading tests was only to guard against a sliding of the points on the hard rock surface. Even though the tests were short duration tests, they indicated satisfactory behaviour under the loading.

Conclusions from loading tests

From the collected loading tests on piles to rock it can, thus, be concluded that a satisfactory behaviour of a pile under a loading can only be ensured provided the pile is supplied with a hardened compact point and is driven until a satisfactory bearing on the rock is established. In the case of inclined rock, the pile has to be driven so carefully that the point does not slide on the rock-surface. The test results show, moreover, that for very long piles it is difficult to ensure a good bearing on inclined rock. Only loading tests with repeated loading and unloading or long term tests will show the quality of the bearing on rock.

The test results in Table IV show, further, that if a steel pile is correctly designed and driven the settlement of the single pile under a loading will be small, approximately equal to the elastic compression of the steel piles. The satisfactory behaviour of the steel piles for the bridge in Fredrikstad, which were designed and driven with full utilization of the available experience, illustrates, thus, clearly a successful application of steel piles under difficult foundation conditions.

SETTLEMENTS OF BUILDINGS ON STEEL PILES

While it is useful to carry out loading tests and study their results, all factors which control the behaviour of the final foundations are not included. A test pile is often driven as a single pile under special control, whereas the piles in the foundation have to be driven in great number without possibilities of "individual" treatment added. To this comes the time effect, which is not included in a short duration loading test. The effect of reconsolidation of the clay around the piles, and of possible regional settlements will contribute to larger settlements than those observed in a loading test. In order to be able to estimate the future settlements of buildings on steel piles, it is, therefore, necessary to study experiences from existing buildings.

Settlement observations from some buildings on steel piles to rock are collected in Table V. In the table the name and height of the building, year of construction and a small situation plan with reference points are given. Moreover, a description of the subsoil and of the rock including a statement of the inclination of the rock surface is also shown. The following columns give the type and length of the pile and the applied stresses.

Concerning the settlements, on only a few of the buildings have level surveys been made since construction. The directly observed settlements, which are quoted in Table V, represent, therefore, only a part of the final, total settlements. It has, thus, been necessary to estimate the total settlements and this has been done from a

TABLE V—SETTLEMENTS OF BUILDINGS ON STEEL PILES TO ROCK.

Build- ing No.	Location	Date of Construc- tion	Sketch Plan	Ground Conditions	Nature of Rock and Inclination	Pile Type, Pile Size	Description of Driving	Pile Length in Feet	Allowable Stress in Pile lb./sq. in.	Settlements		Remarks
										Observed Settlement in.	Estimated Total Settlement in.	
1	Salisbury, at street Hyatt's 7, Oak	1932		Clay	Hard Rock Slight Inclination	2 Welded L's 6" x 6" x 1/2" Rundled Cast Iron Pile 6.474'	No Special Difficulties	① 67-98 ② 72-82 ③ 66-70	17,000	1934-41 ① 0.0' ② 0.2' ③ 0.5'	0.6'-1.4' (Est./Obs. = 1.5:3)	
2	Apartment Block, 53 Baker St., Oak	1932		Filling to 13-19 feet over Clay	Hard Rock Inclination: ① 10° ② 45° ③ 70° ④ 25°	2 Welded L's 6" x 6" x 1/2" H Section Cast Iron Pile 6.474'	Difficult driving, Sinking along Rock Surface Inclination Hard Driving	① 75 ② 45 ③ 59 ④ 33	17,000	1936-55 ① 0.9' ② 3.4' ③ 2.0' ④ 11'	7.2'-4.7' (Est./Obs. = 3:10)	Cracks
3	Apartment Block, 53 Baker St., Oak	1933		Shallow Filling, over Clay Same Ground of Rock Level	Hard Rock Inclination 50°-45°	2 Welded L's 6" x 6" x 1/2" Rundled Cast Iron Pile 6.474'	Difficult to Anchor Filling in same Places Hard Driving	① 49-89 ② 42-78 ③ 72-78	17,000	1938-1948 ① 0.5'-0.7' ② 0.9'-1.5' ③ 0.6'-2.2'	0.8'-3.0' (Est./Obs. = 3:8)	Cracks
4	Milk Room, 33 Schwarzburg St., Oak	1935		Clay	Hard Rock	2 Welded L's 6" x 6" x 1/2" Rundled Cast Iron Pile 6.474'	No Special Difficulties	175-184	11,400	1937-1938 ① 1.4' ② 0.7' ③ 1.1'	1.4'-2.0' (Est./Obs. = 7:3)	
5	Theo. Dahl, Sunderland	1938		Clay	Hard Rock Inclination 30°-45°	2 Welded L's 6" x 6" x 1/2" H Section Cast Iron Pile 6.474'	No Special Difficulties	154 (max 174)	11,400	1938-1953 0.4'-0.6'	0.8'-1.2' (Est./Obs. = 2)	
6	Office Block, Baker St. - corner of O., Oak	1938/39		Clay Sandlayers and Strata of Depth	Hard Rock Inclination 30°-50° with occasional bedges	2 Welded L's 6" x 6" x 1/2" H Section Cast Iron Pile 6.474'	Slight Difficulties in Driving, Slight Inclination on some Piles	① 72 ② 64	11,400	1939-1950 ① 0.1'-0.4' ② 0.7'-1.0'	0.6'-1.4' (Est./Obs. = 3:4)	General Ground Subsidence from 1946 to 1950, approx. ③ called 1'-4" (Observed)
7	Office Block, Baker St. - corner of O., Oak	1938/39		Clay Ground and Sand of Depth	Hard Rock Inclination 30°-45°	2 Welded L's 6" x 6" x 1/2" H Section Cast Iron Pile 6.474'	No Special Difficulties Curvature on Horizontal Axis	① 30 ② 148	11,400	1939-1948 ① 0.3'-0.7' ② 0.9'-1.0'	0.6'-1.4' (Est./Obs. = 3:4)	
8	Old School Building Faneuil Hall 7, Oak	1930		Clay Sandlayer covering Rock	Hard Rock Inclination Horizontal	2 Welded L's 6" x 6" x 1/2" H Section Cast Iron Pile 6.474'	No Special Difficulties	① 92 ② 115	11,400	1950-1955 ① 0.5' ② 0.6'	1.2'-2.0' (Est./Obs. = 3:4)	An Older Adjacent Building experiences large Settlements, by 1945-1955, 1 1/2'-2 1/2'
9	Youth Hostel, Baker Street 7, Oak	1953		Clay	Hard Rock Inclination 40°-45°	2 Welded L's 6" x 6" x 1/2" H Section Cast Iron Pile 6.474'	No Special Difficulties	1 On Rock 2, 33-40 3, 33-50	11,400	1954-1955 ① 0 (None) ② 0.1' ③ 0.1'	0-0.2' (Est./Obs. = 1.5)	
10	Hotel Wing, Malabar Cornerway 3, Oak	1950		Clay	Hard Rock, Slight Inclination	2 Welded L's 6" x 6" x 1/2" H Section Cast Iron Pile 6.474'	No Special Difficulties	1, 40 2, 62 3, 66 4, 33	8,500	1955-1956 ① 0.2'-0.3' ② 0.2'-0.3' ③ 0.2'-0.3'	?	

Table V

study of the time-settlement curves and—if not included in the observations—a calculation of the initial settlements due to the elastic compression of the piles.

It should, however, be noticed at once that the observations which are collected in Table V cannot be said to be representative of the normal Norwegian buildings on steel piles. The majority of the buildings in Table V are characterized either by the fact that they were constructed at an early time in the development of steel piles in Norway, or they are buildings which showed special difficulties

during the pile driving. In such cases the responsible engineers were interested in following the settlements, and settlement observations were made. Of the great majority of buildings which were constructed without difficulties, no settlements were observed. This means that the buildings in Table V are to a higher degree representative of buildings which should be expected to show larger settlements than the majority of normal buildings on steel piles.

For a discussion of the results of the settlement observations, the buildings in Table V may preferably be divided into two groups. The first group includes buildings numbers 1, 4, 5, 8 and 9 and these buildings can be characterized as constructed under normal foundation conditions with no special difficulties during the pile driving. The other group, buildings numbers 2, 3, 6 and 7, are built on sites with difficult soil and rock conditions, i.e., steep inclination of the rock surface, hard layers above rock, etc.

It can be seen at once that the settlement of the buildings in the first group are small and not dangerous. The settlements vary from a quarter of an inch up to two inches, depending on the depth to rock.

Of special interest is building number 1, Folketeateret, Oslo, since this was the first important building on steel piles. Moreover, there are careful settlement observations from the construction until today. Taking into consideration the relatively large stresses, 17,000 lb/in², which were used in the design of the piles, it must be said that the settlement observations have proved that the first application of steel piles in Oslo was very successful.

Another building which is of interest to study is number 8. This is a grain elevator and for such buildings, in contrast to normal office buildings, the design loads will actually occur over a major part of the time. The settlements are also interesting to study for another reason. At the site of the grain elevator it is observed that settlements of the terrain surface are occurring due to ground water lowering, recent fillings, etc. Added to this, the grain elevator is constructed adjoining an existing heavy building founded on a concrete slab. This building has settled one to two feet and in the period since the grain elevator was constructed the settlements amount to 2-3 inches. If one considers the danger of obtaining additional loads in the piles from adhesion of the surrounding clay during the settlements, it is of interest to note that the total settlements of the grain elevator are

only 1-2 inches and the smallest values are observed on that side which is connected with the building with large settlements.

In the group of buildings with difficult piling conditions, numbers 2 and 3, the Bislet Buildings are in a particular class. Both buildings are among the very first on steel piles in Norway and it was, actually, on these sites that Norwegian contractors and consulting engineers for the first time were faced with difficult driving conditions of steel piles. At Bislet IV the rock surface was inclined as much as 70° and the pile points slipped on the rock surface. Both the pile driving and the control were characterized by lack of experience. At Bislet V the conditions were further complicated by the great depths to rock and to this there were added hard layers with boulders above the rock. Both buildings show considerable settlements. The total settlements amount to 4 to 5 inches with the differential settlements 3 to 4 inches, which, of course, have resulted in cracking and increased maintenance costs.

It is obviously of interest to compare the Bislet buildings with the two office buildings—number 6 and 7—at Fridtjof Nansens Plass in Oslo. The conditions are almost equal for both buildings and they are characterized by their extreme difficulties from the point of view of pile driving. The buildings are erected on the edge of a clay-filled canyon with almost vertical rock faces. Fig. 17 shows a photograph of a model of one of the buildings including the pile foundation. As can be seen, one side of the building is founded directly on rock by 15-20 foot deep concrete piers. The opposite side of the building is carried by 150 foot long steel piles. In the design of the steel pile foundation, the fact that the rock surface is interrupted by a few narrow terraces was utilized and the points of the piles were driven down to these terraces in order to prevent slipping of the points.

The buildings at Fridtjof Nansens Plass were constructed 6 years later than the Bislet buildings and much experience had been collected by the extensive use of steel piles during this period. This fact may explain why the pile driving for the buildings at Fridtjof Nansens Plass was carried out successfully and with only a few rejected piles. As seen from Table V, the settlements of the buildings are small and not dangerous. They only amount to a maximum of one and a half inches and this in spite of the fact that the area around the buildings shows serious general subsidence.

The experience from the buildings at Fridtjof Nansens Plass

are a clear demonstration of the possibilities which steel piles present for the utilization of sites with extreme foundation difficulties.

In order to get an evaluation of the time effect on the settlements, a comparison between settlements observed on buildings and during loading tests of single piles is of interest. In Table IV, the

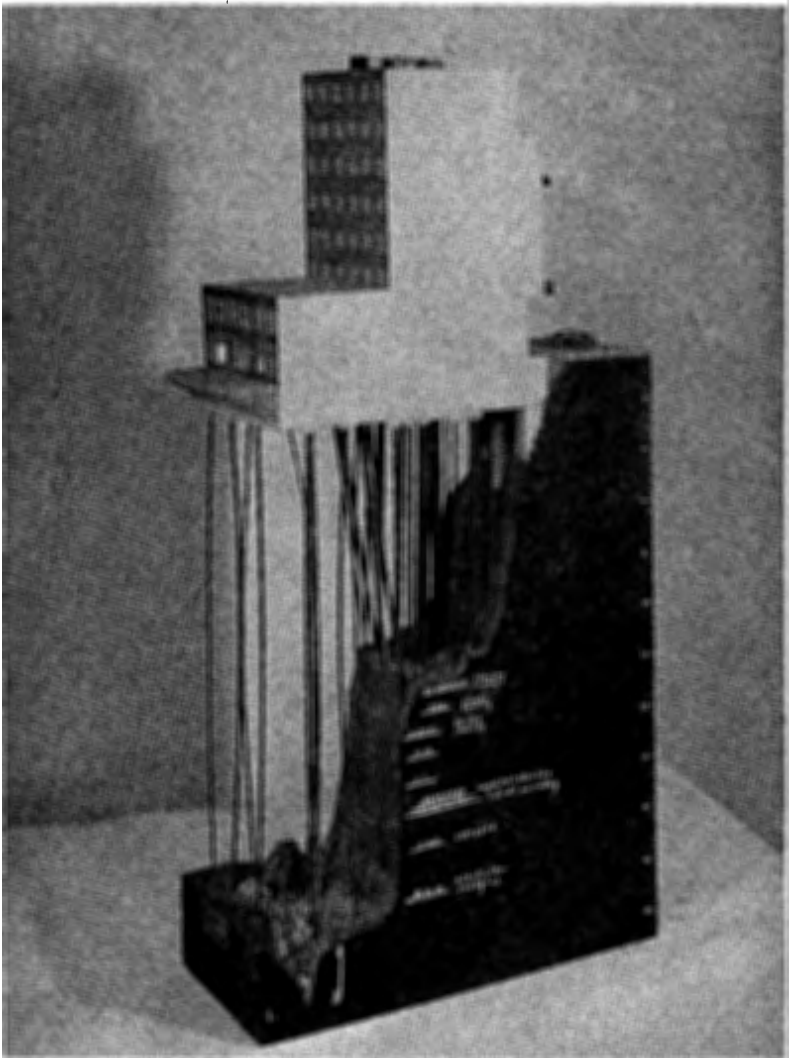


FIG. 17.—MODEL OF PILE FOUNDATION OF FRIDTJOF NANSENS PLASS NUMBER 8.

settlements observed by loading tests are compared with and found approximately equal to the elastic compression of the piles. It is directly seen from the settlement observations on the buildings that the total settlement of the buildings are in all cases larger than the elastic compression of the piles. In Table V in the column with the total settlements, in brackets, the ratio of the total settlements of the buildings to the elastic compression of the piles is quoted. For the calculation of the elastic compression of the piles the probable stress in the piles was used; this means that the loads on the piles under office buildings have been reduced by 20-30 per cent. As directly seen from the table, the ratio of the settlements to elastic compression of the piles is fairly constant. Except the two Bislet buildings, which, as mentioned, are in a special class, the ratio varies between 2 and 4. This means also that the settlements of the buildings are 2 to 4 times the settlements found by a loading test.

The explanation of the larger settlements of buildings compared with those of single piles is probably partly due to negative skin friction on the piles resulting from the reconsolidation of the surrounding clay, which was remoulded during the pile driving, and partly it may be due to creep in the steel and to local deformations of the rock below the pile points. Finally, it may be that small upheavals of the piles have taken place after the pile driving. For most of the buildings this effect was not fully compensated for by re-driving of the piles. As pointed out above, the buildings in Table V were built at a relatively early time in the development of steel piles in Norway and the importance of a careful securing of the pile points into the rock, and of a systematic re-driving were not always appreciated at that time.

Summarizing the results of the settlement observations, it may be concluded that experiences from existing buildings have clearly demonstrated that, if properly designed, steel piles may be applied successfully to the foundation of the buildings, even under the difficult and varying soil conditions met with in Oslo.

CLOSING REMARKS

In the above review of Norwegian experiences with steel piles to rock the conclusions which can be drawn from the collected observations are given at the end of each section, and as they are concerned mostly with details in design and driving of piles, they will not

be repeated here. It may, however, be reasonable to end this paper with a few remarks of a more general character.

The small-scale work which is done with foundations in Norway is, of course, not comparable with the important foundations carried out for heavy buildings in larger countries with old traditions in foundation engineering. However, the study of the development of steel piles in Norway is an interesting and illustrating example on how technical conquests are initiated by practical engineers through experiments—and very often daring experiments. The example demonstrates, moreover, that the daring pioneer work is only changed into a true development and a substantial progress provided they are succeeded by careful studies of the behaviour of the finished structures. In the case of foundations in Norway these factors—the daring experiments followed by careful studies of the finished buildings—are the main contribution to the existing tradition in foundation engineering.

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