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NORWEGIAN EXPERIENCE WITH STEEL PILE FOUNDATIONS TO ROCK*

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(Presented at a meeting of the Structural Section, B.S.C.E., held on March 13, 1957.)

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

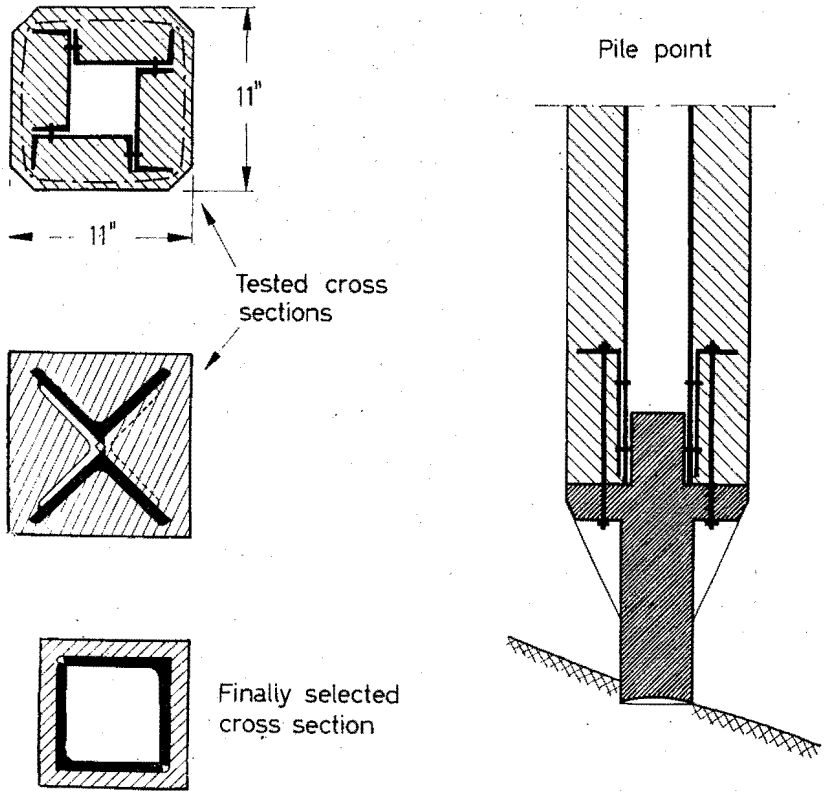


FIG. 1.—CROSS SECTIONS OF STEEL PILES AND PILE POINT USED FOR FOLKETEATERET OSLO 1931. (FOLKESTAD 1934).

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-

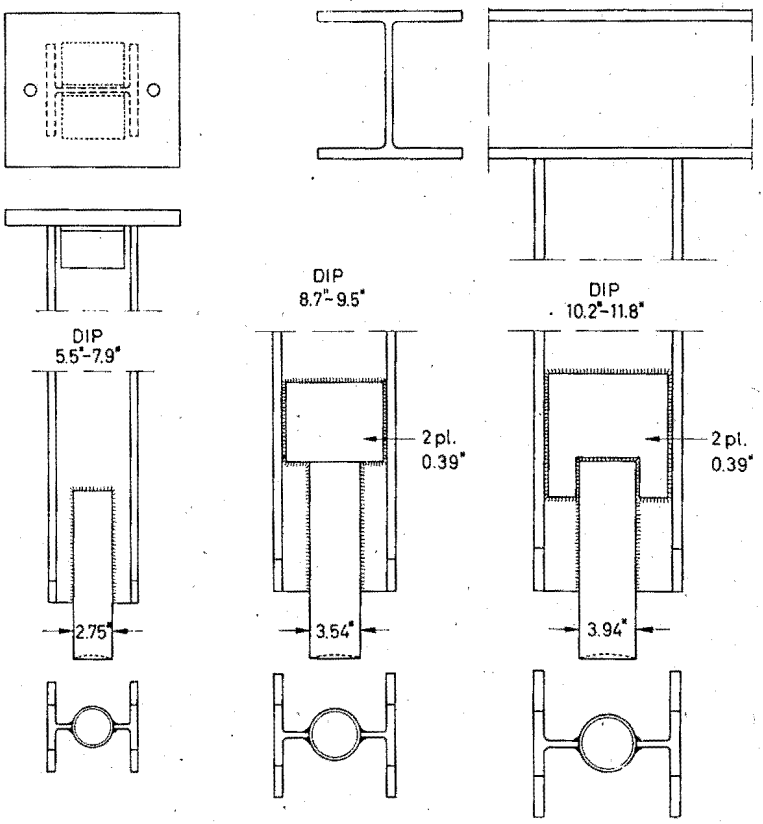


FIG. 2.—H-SECTION STEEL PILES WITH POINTS AND CAPS AS REQUIRED BY THE BUILDING AUTHORITY OF OSLO.
DIP is the notation of a wide-flanged H-beam in the Deutsche Industrie-Normen (DIN 1025).

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

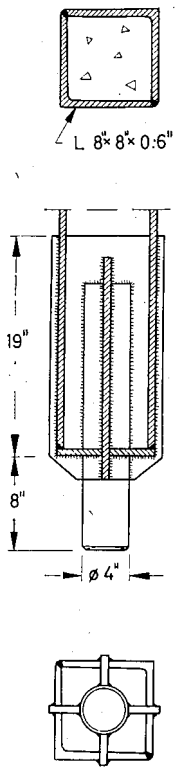


FIG. 3.—BOX PILE USED UNDER DIFFICULT FOUNDATION CONDITIONS.

stress 17100 lb/in^2. 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/in^2 are permitted for piles with a length smaller than 40 feet. For pile lengths from 40 to 100 feet, 11400 lb/in^2 is allowed and for piles which are longer than 100 feet the allowable unit stress is reduced to 9200 lb/in^2.

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 Institute 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.

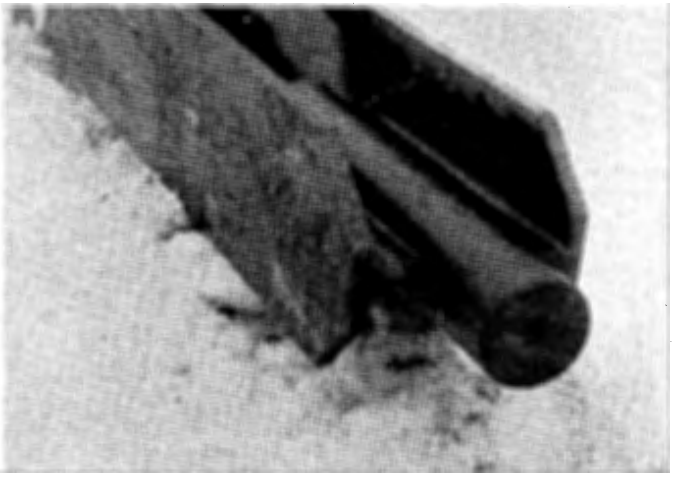


Fig. 4.—Photograph of Pile Point for H-Section.

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

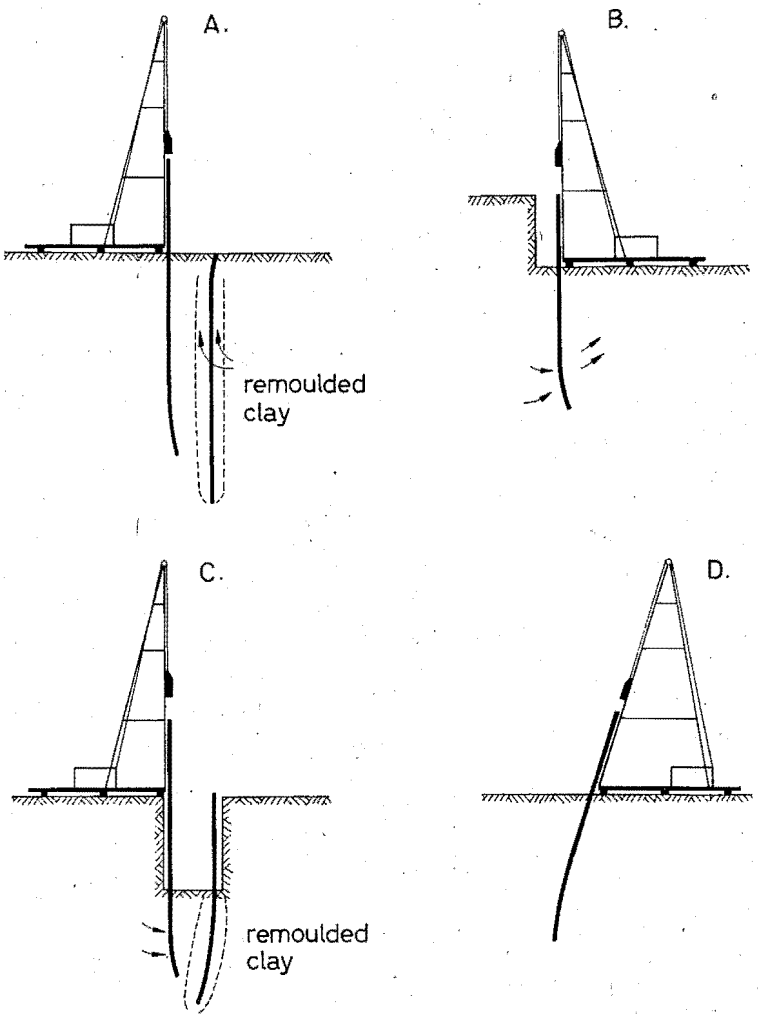


FIG. 5.—FACTORS WHICH CAUSE MISALIGNMENTS OF PILES DRIVEN INTO SOFT CLAY.

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 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.

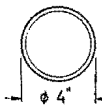
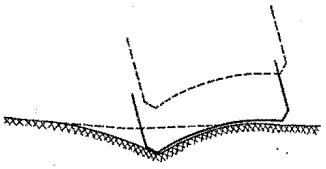
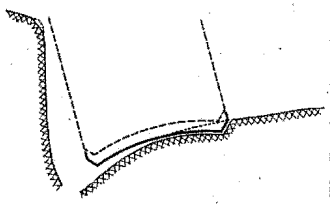
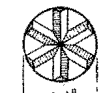
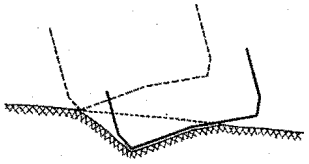
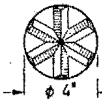
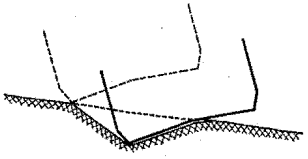
Pile no. Type of point	Bearing on rock	Pile driving data		
		Number of blows	Height of ram, inches	Settlement inches
Pile no. 1  φ 4" Oslo-point		100 100 100 100 100 100 100 100	4-8 8 8-10 8-10 8-10 8-10 28 28	0.20 0.55 0.24 0.12 0.04 0.04 0.08 0
		800		1,27
Pile no. 2  φ 4" Oslo-point		100 100 25 5	4-6 12-14 24-28 47	3.5 0.5 0.08 0.04
		230		4,12
Pile no. 3  φ 4" Corona-point		100 100 100 100 100 5	4-6 8 10-12 16 24 47	0.24 0.04 0.20 0.08 0.16 0.04
		505		0,76
Pile no. 4  φ 4" Corona-point		100 100 25 5	4-6 20 40 50	0.04 0.20 0.08 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.

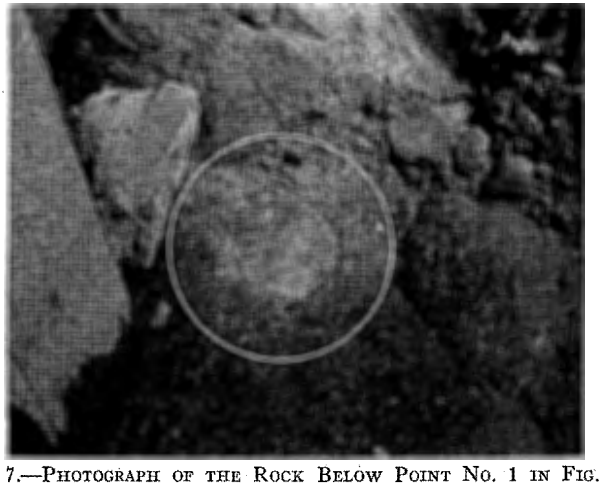


FIG. 7.—PHOTOGRAPH OF THE ROCK BELOW POINT NO. 1 IN FIG. 6. The circle indicates the depression caused by the pile point.

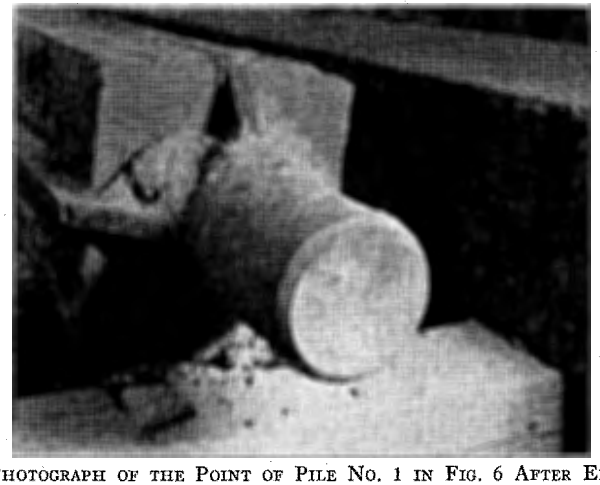
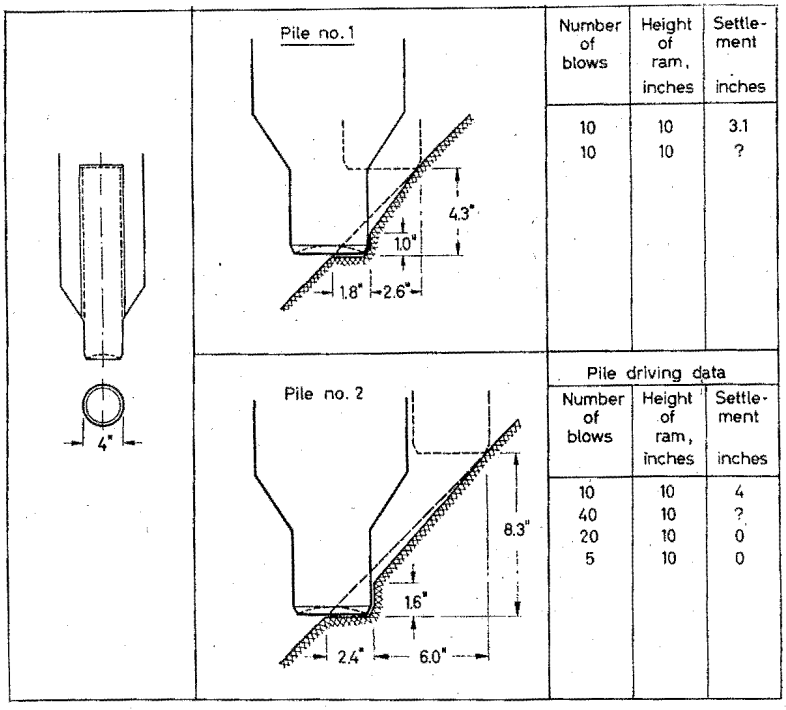


FIG. 8.—PHOTOGRAPH OF THE POINT OF PILE NO. 1 IN FIG. 6 AFTER EXCAVATION.

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



Number of blows	Height of ram, inches	Settlement inches
10	10	3.1
10	10	?

Pile driving data

Number of blows	Height of ram, inches	Settlement inches
10	10	4
40	10	?
20	10	0
5	10	0

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 ¾″ Ø 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 clay[1]. 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/in^2. For typical values of length and width of steel piles, c will vary from 100 to 600 lb/in^2.

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,

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

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

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

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.

[1]The 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 $\sigma_{max} = 52000$ lb/in^2 and $E = 3 \times 10^7$ 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

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

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
I	Train rails	0.17
I	H-sections 8"× 8"	0.31
I	H-sections 10"×10"	0.34
○	Steel tubes ø 12" δ = 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 1½″ 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 1¼ 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

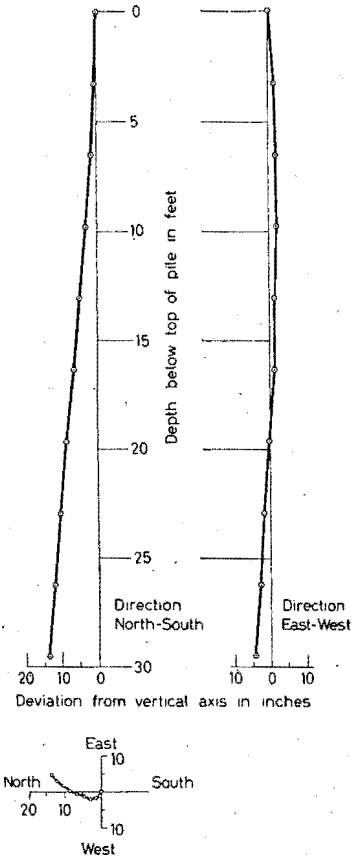


FIG. 10.—RESULT OF MEASUREMENT OF THE CURVATURE OF A STEEL PILE.

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 to rock feet	Pile length in clay feet	Loading tests P_{max} tons	Loading tests σ_{max} lb/in²	Loading tests δ_{max} in	Loading tests Buckling length	Estimated k lb/in³	Calculated critical buckling length	Calculated buckling load (initially straight pile) tons
2	● ø 1¼ in	118	79	9	18 300	c 0.9	4' 3"	110	3'4"	16.3
3	A = 1.1 in²	121	79	9	18 300		6'5"	110	3'4"	16.3
4	I = 0.096 in⁴	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 in² I_{min} = 8.4 in⁴	c 118	79	150	49 400	2.9	?	110	10'1"	153
7	I h = 4.5 in b = 4.7 in A = 6.7 in² I_{min} = 4.8 in⁴	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/in^2. 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/in^2 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/in^2 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]

[2]The 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 1¼″ 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.

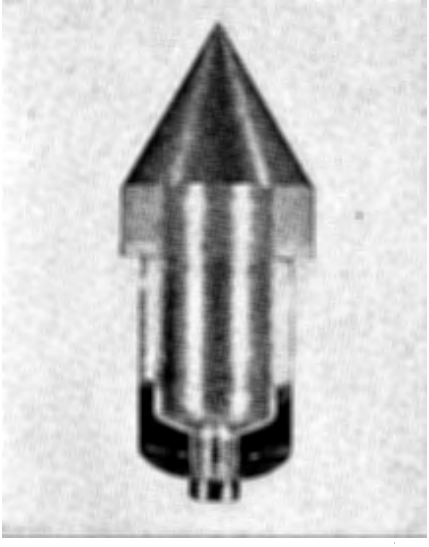


FIG. 11.—PHOTOGRAPH OF THE CORROSION POINT FOR IN SITU MEASUREMENTS OF CONDUCTIVITY AND DEPOLARIZATION OF SOIL.

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.

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

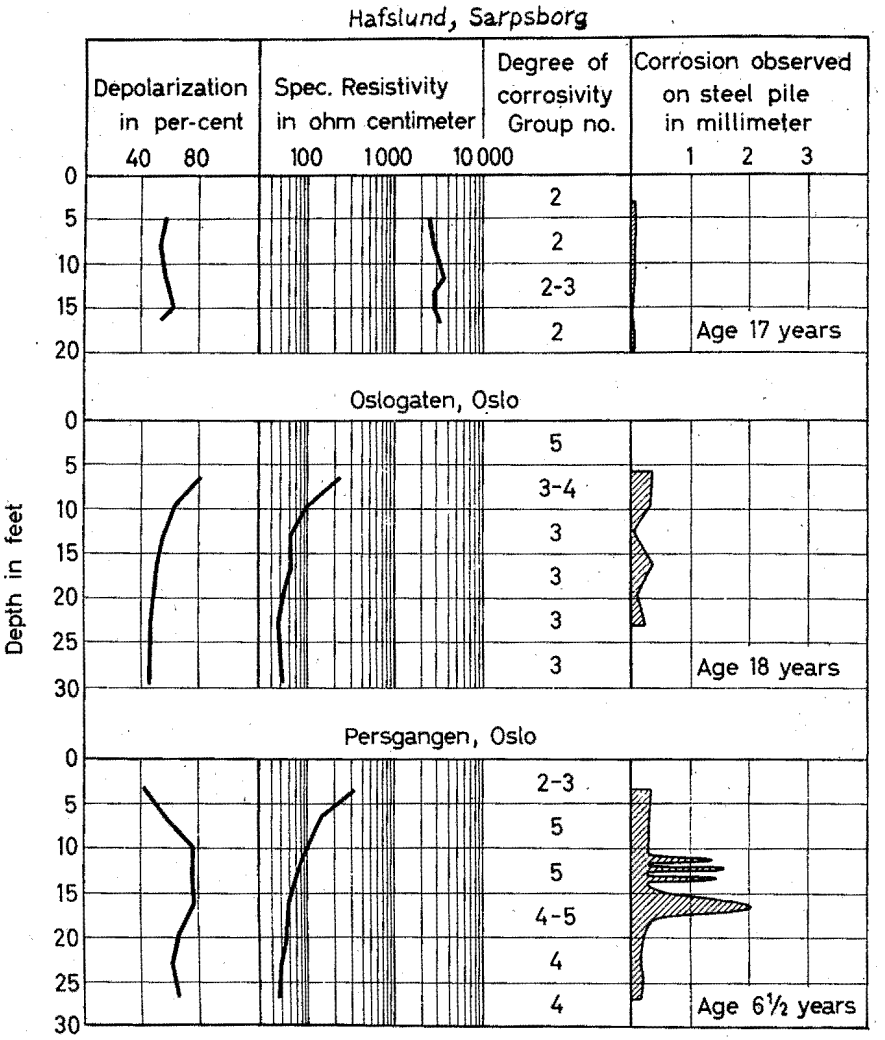


FIG. 12.—EXAMPLES ON RESULTS OF IN SITU MEASUREMENTS WITH THE CORROSION POINT AT THREE DIFFERENT SITES, WHERE ACTUAL CORROSION WAS OBSERVED ON STEEL PILES.

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: Cross-Section	Pile Type and Dimensions: Point	Length Feet	Ground Conditions: Soil	Ground Conditions: Rock	Driving: Penetration after Contact	Loading Tests: Loading Programme	Loading Tests: Duration	P_{max} tons	σ_{max} lb/in²	δ in	δ/D Value	Permanent Set in	Remarks
1	Frickstrom 1913 (Forsell 1918)	Reinforced Concrete 10"×10"	Steel Point	46	Sandy Clay	Hard, horizontal		0-30-0	30 tons for 24 hours	30	570	0.0"	0.0	0	
2	Skansen, Gothenburg 1930 (John Dixson)	● ø 3¼" A = 0.44 in²	No Point	20	Clay	Granite		0-3-0	3 tons for one month	3	14 900	0.3"	6	0.1"	Failure at 4 tons, i.e. 19 900 lb/in²
3	Sørenga, Oslo 1930 (F. Selmer)	9½"×9½"×50 lb H Section A = 13.2 in²	No Point	102	Clay	Horizontal	4000 lb Ram 6'-6" fall Height	0-200-0	200 tons for 30 mins	210	27 000	1.2"	1.2	0.5"	
4				103						210	27 000	1.6"	1.6	0.6"	
5				105						210	27 000	1.5"	1.4	0.5"	
6		10½"×10½"×60 lb H Section A = 17.4 in²		105			3'-3" Fall Height			210	20 300	1.4"	1.4	0.5"	
7				105						210	26 300	1.1"	1.0	0	
8				105						200	24 800	1.6"	1.6	0.5"	
9	Folleskolehet, Oslo 1931 (Fæstad 1934)	4 ⌐⌐ 6¼"×2½"×12.7 lb A = 14.9 in²	Cast Iron Point ø 4"	482	Clay	Hard, slight Inclination				180	25 500	0.8" approx	c 1.0	0	
10	Göthenburg 1940 (Gefverson 1941)	6¼"×6¼"×49 lb H Section A = 12.2 in²	Oslo Point ø 3"	56	Clay	Hard granite	2 tons Ram 5-0 after Contact		Short Duration	190	24 400	0.6"	0.9	0	Undisturbed after Pulling
11				98			5-0			97	23 700	1.0"	0.9	0	
12				154		85° Inclined	5-5'6"			80	27 000	1.9"	0.9	0	Sharp Bend, 5'-0" from Point after Pull
13		8⅛"×8⅛"×48 lb H Section A = 14.1 in²	Oslo Point ø 4"	80		Hard granite, horizontal	After Contact 400 blows 2 tons × 1'-8" δ = 3.1"	0-100-0-150-0	100 tons for 2 hours	100	15 600	0.5"	0.9	0	
14				105		c 45° Inclined	After Contact 14 blows 2 tons × 1'-8", δ = 3'-0"	0-100-0-120-0	110 tons for 65 hours	110	18 500	1.2"	1.0	0.6"	
15	Fragerveiskje, Oslo 1940 (F. Selmer)	Reinforced Concrete 10"×10"	Steel Point ø 4"	85	Clay	Hard		0-167-0	167 tons for 17 months	167	2 500 approx	1.7"	1.5	0.2"	Pile Joints Inside Steel Casing
16	Boston 1946 Cosgrove 1947	14"×14"-76 lb H Section A = 22.2 in²	No Point	144	Clay	Slate horizontal	δ = ø 0.1 in/blow	0-100-200-150-0	160 time months 330 tons for 5 hrs	200	19 000	0.5"	0.8	0.1"	
17	Stenmestbol 1940 (Bernhon 1955)	8"×8"×41 lb H Section A = 12.0 in²	Oslo Point ø 4"	13	Clay	Alum slate c 35° Inclined	δ = 2" after Contact Slid 1'6" approx	0-40-90	90 tons for 4 mins	90	15 600			0.2"	Failure at 80 tons
18				16		c 60°	Slid 8" approx	0-40-90		90	15 600			5.0"	
19	Nordsjøing 1953 (Sunstein 1954)	4¼"×4¼" A = 27.3 in²	Oslo Point ø 4"	115	Clay Gravel	Granite, slight Inclination	No appreciable Penetration after Contact	Repeated loading and Unloading		180	17 700	0.8"	1.0	0	
20		8⅛"×8⅛"×48 lb H Section A = 14.1 in²		115						180	18 400	0.8"	1.0	0	
21	Uddevalla 1954 (Grønholm)	Reinforced Concrete 11¾"×11¾" A =	Oslo Point ø 3¼"	93	Clay	Horizontal	After Contact 250 blows with 4 tons × 2'-6" δ = 0.025"/blow	Repeated loading and Unloading	Short Duration	134	1 940 approx	0.7"	1.0	0	
22	Lademoen Church Trondheim 1956 (A. Brandtzæg 1955)	4¼"×4¼" A = 5.7 in²	None	115	Gravel-Clay Gravel	Relatively horizontal	2 700 lb Ram	0-115-0-120-0	Short Duration	120	35 700	1.6"	0.3	0.1"	Representative Pile
23				105-121						100-120	33 700-30 700			0.0"	Average of 15 Piles
24				115		Very rocky Driving		4 loading and Unloading Cycles		120	39 700	3.7"		4.4"	Failure of First loading, 150 tons approx
25	Fredrikstad 1955	8"-8" × 33 lb Welded L's A = 10.2 in²	Oslo Point ø 4"	85	Clay	Hard granite, horizontal	5000 lb Ram Careful Driving, Approx 1000 blows after Contact	0-200-0-200-0	100 tons for 600 hours	200	22 700	0.8"	1.0	0	
26				69		Slight Inclination	δ = 1.4" after Contact approx 900 blows	0-200-0	200 tons for 20 hours	200	22 700	0.7"	1.1	0.2"	Straight before Test After Test Radius of Curvature = 13 000 Feet
27				72			δ = 1" after Contact	0-200-0	Short Duration	200	22 700	0.5"	0.9	0	Raking Pile, d =
28				65			δ = 0.4" after Contact	0-174-0	174 tons for 4 hours	174	20 000	0.6"	1.1	0.1"	Radius of Curvature R = 3 300 Feet
29	New York 1939 (Glick 1948)	15"×14"-109 lb H Section A = 29.7 in²		188	100 ft soft Clay and Silt 20 ft soft Silt and Sand 53 ft Sand + Gravel		14 000 lb Ram 2'-3" fall Height Final Penetration = 0.02/blow	0-200-0		200	15 100	0.9"	0.8	0.2"	
30	New York 1947 (Glick 1948)	12"×12"-53 lb H Section A = 15.6 in²		70	Silt and fine Sand		6 500 lb Ram 3'-8" fall Height Final Penetration = 0.12/Blow	0-150-0		150	10 500	0.60"	1	0.16"	
31		14"×14"-73 lb H Section A = 21.5 in²		80						150	15 000	0.35"	0.7	0.05"	
32		14"×14"-89 lb H Section A = 26.2 in²		107						150	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:

Sørenga, 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 Sørenga 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

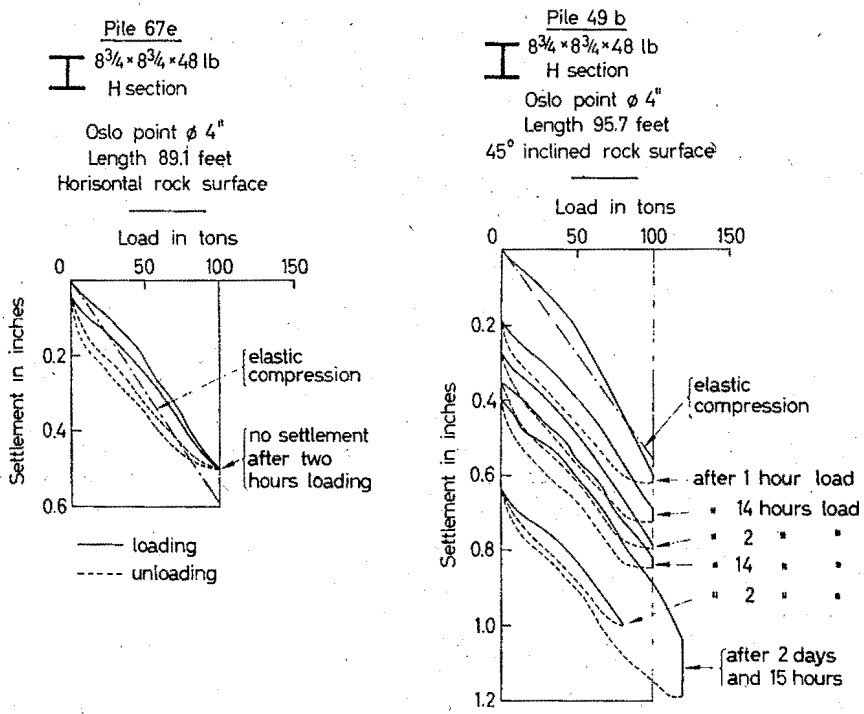


FIG. 13.—RESULTS OF LOADING TESTS WITH TWO STEEL PILES IN GOTHENBURG (PETTERSON 1941).

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 ¾ 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

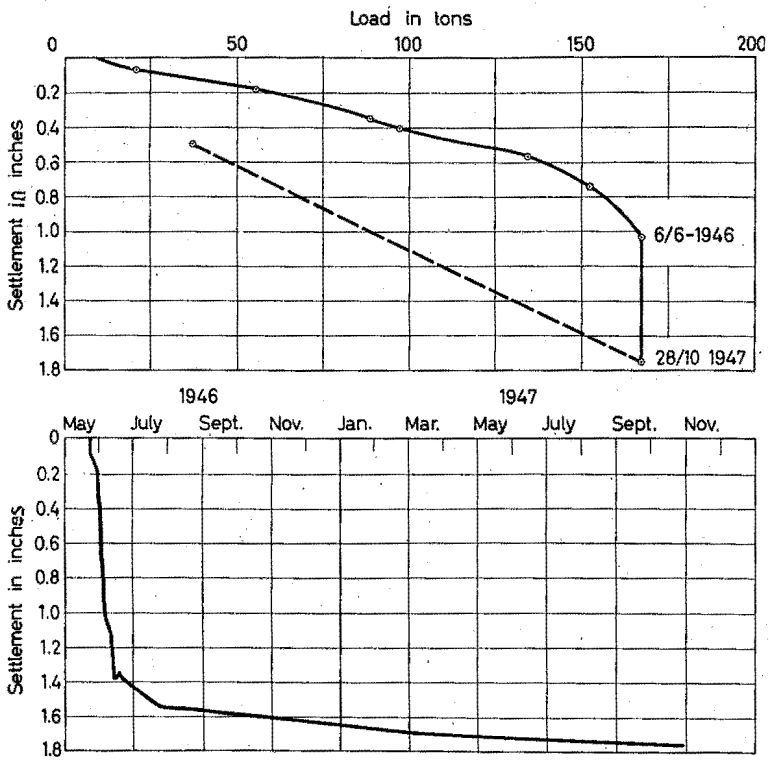


FIG. 14.—RESULTS OF 17 MONTHS LOADING OF A 87 FEET LONG CONCRETE PILE IN OSLO. THE PILE, WHICH WAS SUPPLIED WITH A 4" DIAMETER OSLO-POINT, WAS DRIVEN APRIL 23-24, 1946.

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

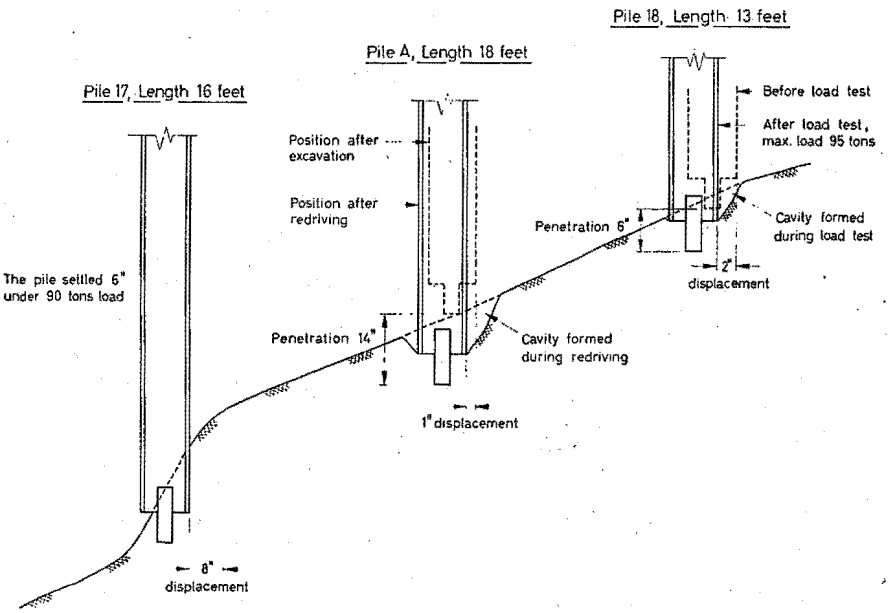


FIG. 15.—STEEL PILES IN SLEMMESTAD EXCAVATED AFTER THE DRIVING (BERNTSEN 1955).

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/in^2. 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 stepwise 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 ¾ 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.

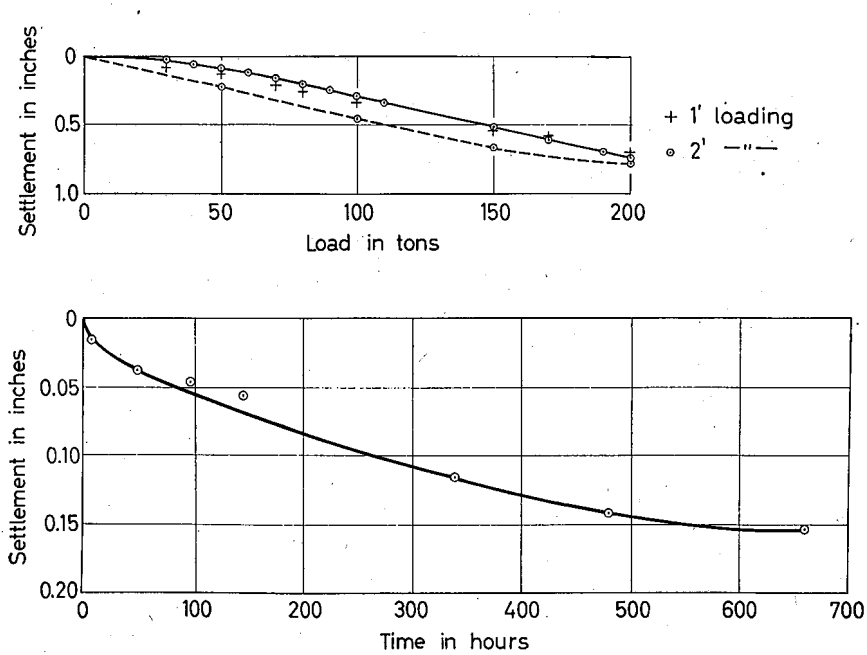


FIG. 16.—LOADING TEST ON A 85 FOOT LONG STEEL PILE TO ROCK AT THE BRIDGE AT FREDRIKSTAD.

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.

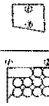
Building No.	Location	Date of Construction	Situation Plan	Ground Conditions	Nature of Rock and Inclination	Pile Type. Pile Point	Description of Driving	Pile Length Feet	Allowable Stress in Pile lb/in²	Settlements: Period and Measured Settlements in	Settlements: Estimated Total Settlement in	Remarks
1	Folkebokhal, 11 stories Nytorget 7, Oslo	1932		Clay	Hard Rock Slight Inclination	? Welded L's 6¼"-6¼"-½" Gunited Cast Iron Point ø 4½"	No Special Difficulties	① 69-98 ② 72-82 ③ 66-70	17.000	1934-41 ① 0.9" ② 0.2" ③ 0.5"	0.6"-1.4" (S_{ob}/S_{cal} = 15-3)	
2	Apartment Block, 4 stories Bislet 17, Oslo	1932		Filling to 13-19 feet over Clay	Hard Rock Inclination: Ⓐ 10° Ⓑ 45° Ⓒ 70° Ⓓ 25°	8"-8"-44 lb H section Cast Iron Point ø 4½"	Difficult driving, Slipping along Rock Surface Relatively Hard Driving	Ⓐ 75 Ⓑ 46 Ⓒ 59 Ⓓ 33	17.000	1936-55 Ⓐ 0.9" Ⓑ 3.4" Ⓒ 2.8" Ⓓ 1.1"	12"-47" (S_{ob}/S_{cal} = 3-10)	Cracks
3	Apartment Block, 5 stories Bislet 17, Oslo	1933		Shallow Filling, over Clay Some Gravel of Rock level	Hard Rock Inclination 30°-45°	? Welded L's 6¼"-6¼"-½" Gunited Cast Iron Point ø 4½"	Difficult to Penetrate Filling in some Places Hard Driving	① 49-88 ② 82-118 ③ 82-98	17.000	1938-1946 ① 0.3"-0.7" ② 0.9"-1.5" ③ 0.6"-2.7"	0.8"-3.9" (S_{ob}/S_{cal} = 3-8)	Cracks
4	Milk Depot, 3 stories Schweigaardsgt 35, Oslo	1935		Clay	Hard Rock	? Welded L's 6¼"-6¼"-½" Gunited Cast Iron Point ø 4½"	No Special Difficulties	115-154	11.400	1937-1946 ① 1.4" ② 0.7" ③ 1.1"	1.4"-2.0" (S_{ob}/S_{cal} = 2-3)	
5	Thor Dahl, Sandefjord	1938		Clay		8"-8"-44 lb H Sections 9½"-9½"-50 lb Oslo Point	No Special Difficulties	c. 154 (max 174)	11.400	1938-1953 0.4"-0.6"	0.8"-1.2" (S_{ob}/S_{cal} = 2)	
6	Office Block, 8 stories Fr. Nansens pl 8, Oslo	1938/39		Clay Sandlayers and Stones at Depth	Hard Rock Steep Inclination (30°-65°) with occasional ledges	8"-8"-44 lb H Sections 8¼"-8¼"-48 lb 9½"-9½"-50 lb Oslo Point	Slight Difficulties in Driving. Slight Inclination on some Piers	① 72 ② 164	11.400	1939-1950 ① 0.2"-0.4" ② 0.7"-1.0"	0.6"-1.4" (S_{ob}/S_{cal} = 3-4)	General Ground Subsidence from 1941 to 1950, point ③ settled 2"-4" (Observed)
7	Office Block, 8 stories Fr. Nansens pl 9, Oslo	1938/39		Clay Gravel and Sand at Depth	Hard Rock Inclination 30°-65°	8"-8"-44 lb 8¼"-8¼"-48 lb 9½"-9½"-50 lb 10¼"-10¼"-60 lb Oslo Point	H Sections	① 30 ② 148	11.400	1939-1948 ① 0.3"-0.7" ② 0.9"-1.0"	0.6"-1.4" (S_{ob}/S_{cal} = 3-4)	
8	Oslo, Schous Bryggeri A/S Trondheimsveien 7, Oslo	1939		Clay Sandlayer covering Rock	Hard Rock Slight Inclination - Horizontal	7¼"-7¼"-35 lb 8"-8"-44 lb 8¼"-8¼"-48 lb 9½"-9½"-50 lb Oslo Point	H Sections	① 92 ② 115	11.400	1950-1955 ① 0.5" ② 0.6"	1.2"-2.0" (S_{ob}/S_{cal} = 3-4)	An Older Adjoining Building experiences large Settlements, eg 1945-1955, 14"-24"
9	Vestre Hotel, 5 stories Storgt 2, Oslo	1953		Clay	Hard Rock Inclination 40°-45°	8"-8"-44 lb H Sections 9½"-9½"-50 lb Oslo Point	No Special Difficulties	1 On Rock 1 33-49 3 33-59	11.400	1954-1955 1 0 (Piers) 2 0.1" 3 0.1"	0-0.2" (S_{ob}/S_{cal} = 1.5)	
10	Hotel Viking, 14 stories Gunnerusgt 3 Oslo	1950		Clay	Hard Rock, Slight Inclination	8"-8"-44 lb H Sections 11"-11"-76 lb Oslo Point		1 40 2 62 3 66 4 33	8.500	1953-1955 1 0.2" none 2 0.2" --- 3 0.2" --- 4 0.2" ---	?	

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/in^2, 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

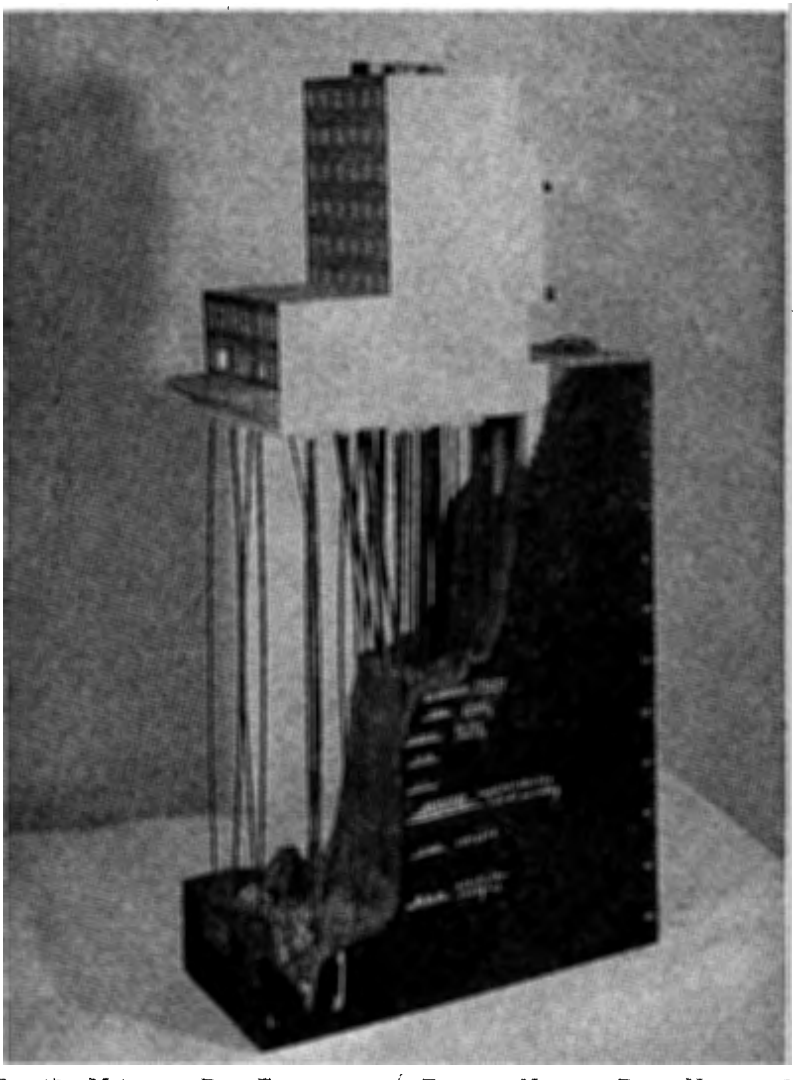


FIG. 17.—MODEL OF PILE FOUNDATION OF FRIDTJOF NANSENS PLASS NUMBER 8.

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 redriving 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 redriving 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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WHAT THE FEDERAL HIGHWAY ACT OF 1956 MEANS TO MASSACHUSETTS

By H. Gordon Gray*

(Presented at a meeting of the Transportation Section, B.S.C.E., held on April 24, 1957.)

Much has been said about the Federal Highway Act of 1956 concerning what it will do, what it will mean to engineers, contractors and material suppliers, and how it will effect the entire nation.

I would like to talk for a few minutes with you on the impact of this Act on Massachusetts and in order to approach this subject intelligently, let me briefly review the Act.

Congress, in passing the Federal Aid Act of 1956 gave full recognition to the urgent need for improvements to the Interstate system and at the same time, provided for the continuation of Federal Aid appropriations for improvement to the Federal Aid primary, secondary and urban highway systems. In providing 24 billion, 825 million dollars for the Interstate system, Congress authorized the largest appropriation ever set up for purposes other than National Defense, and they made provisions for financing the cost.

In arriving at the amount of money which would be needed to complete improvements to the Interstate system, Congress had recourse to an estimate of highway needs which had been prepared by the Bureau of Public Roads, with the cooperation of the 48 states, in accordance with Section 13 of the Federal Highway Act of 1954.

This early needs study, you may recall, estimated that the total Highway needs throughout the nation totaled 101 billion dollars. Of that total amount, 23 billion was estimated as needed for the Interstate system, 45 billion for other Federal Aid systems, 6 billion for other state highways and 27 billion dollars for other roads and streets (either county or city facilities.)

I am bringing this to your attention since many believe that the 1956 Federal Act is the cure-all and end-all of our highway problem. Actually, the needs for improvement of road systems, not a part of the Interstate network, far exceed the cost of needed Interstate im-

*Deputy Chief Engineer for Inter State Highway System, Massachusetts Department of Public Works.

provements, so despite the fact that 25 billion dollars has been authorized to meet the Federal Government's 90% share of Interstate costs, other major highways do not benefit from these funds.

Massachusetts estimate of highway needs made in 1954, showed that 838 million dollars would be required for the Interstate network.

The Federal Aid Act of 1956 provides for the distribution of the Interstate 90-10 money for the first three years 1957, 58 and 59 on the basis of a previously approved formula, which gives weight to population, area and post road mileage. This formula would give Massachusetts 2.22% of all Interstate allocations for these years. For the remainder of the program, that is from 1960 through 1969, the bill provides for the distribution of funds to states on the basis of need.

In order that all states would be computing their needs on the same basis, the Bill provides for a revised needs cost study to be submitted to Congress for their 1958 session. We are presently preparing this estimate, to uniform standards issued by the Bureau of Public Roads, based on an approved location for each of the Interstate routes and at present, we estimate that our new needs requirements will exceed our 1954 estimate of 838 million.

We now have the advantage of the new design standards for the Interstate system and we are in quite considerable detail, now assigning realistic anticipated traffic to the select locations, to determine the number of lanes required, as well as the interchange locations and determining a feasible profile, so that our latest needs estimate will reflect close approximations of earthwork quantities, bridge costs, surface costs, right of way costs, etc.

Now, based on the allocation of funds for the first three years on the old formula basis, Massachusetts will receive a total of 100 million 300 thousand dollars for the Interstate system which, when matched with 10% state funds will finance a program of 111 million dollars during that period.

Assuming that the succeeding allocations of Federal Aid Interstate funds will be made in the ratio of our 1954 needs estimate, that is, 838 million to the National total of 23 billion, we should receive 3.6% of all fiscal allocations beyond 1960 or about 79 million dollars each year through 1967 when the yearly total allocations are diminished for the last two years.

This 79 million dollars representing 90% of a total program and

when matched with roughly 9 million dollars of state funds, the 10% portion, will provide for a continuing program of 88 million dollars per year for the Interstate system alone.

In summary, over the 13 year period we will have an 838 million dollar program of Interstate projects and assuming that our regular Federal Aid funds for primary, secondary and urban systems continue at the rate of 16 million dollars a year, a program of 420 million dollars will exist for these systems.

Averaged out for the 13 year period, we will be executing a Federal Aid program of about 100 million dollars per year for the entire period and the state's share of this total will be roughly 300 million.

Now for further repetition, let me tell you what the Interstate system is. Congress in 1944 authorized the selection of a special network of highways not to exceed 40,000 miles in length, to connect by routes, as direct as practicable, the principal metropolitan areas, cities and industrial centers, to serve the National Defense and to connect at suitable border points with routes of continental importance in Canada and Mexico.

The result was the creation of the National system of Interstate Highways embracing only 1.2% of the Nation's total road mileage, but joining 42% of the state capitols and 90% of all cities over 50 thousand population. 37,600 miles were first allocated and in 1955 the remaining 2400 miles were allocated to urban extensions in the various states.

Massachusetts now has 415 miles of highway in the Interstate system, 70 miles having been added in 1955 as urban extensions including the Boston Inner Belt, the Worcester Inner Belt, the Springfield Expressway and an outer Belt, about 10 to 15 miles outward of route 128 which would extend from Route 1 in Foxboro to Route 1 at the New Hampshire line in Salisbury.

Our system as it now exists is made up of the following routes:

Route 1—From Rhode Island state line to New Hampshire.

Route 5—From Connecticut state line to Vermont state line.

Route 6—From Rhode Island to New Bedford.

Route 15—From Connecticut state line to Sturbridge, where a connection is made to the East West Toll Turnpike.

Route 28—From Boston to the New Hampshire state line at Methuen, and as mentioned above, the Boston Inner Belt, the Worces-

ter Inner Belt, the Springfield Expressway and the so called outer belt.

The East West Toll Turnpike has tentatively been included in the Interstate system since it will meet the high Interstate design standards and traffic volumes would not warrant the construction of a parallel free route serving the same areas. Under the provisions of the Act, however, no reimbursement can be claimed for the cost of constructing the East West Toll Turnpike, since it is a Toll facility.

Inasmuch as the so called turnpike extension from Route 128 in Boston had not been constructed, the Department has included this portion of the facility in its estimate of needs for the Interstate system, so that, in the event the Turnpike Authority feel that they are not in a position to construct this extension, the Commonwealth will be able to construct this with 90% Federal funds.

Prior to the passage of the 1956 Act, Massachusetts had under its own accelerated program, brought certain sections of the Interstate system up-to-date. The principal sections are 21 miles of Route 1 from Danvers to the New Hampshire state line, the Central Artery portion of the Boston Inner Belt, a large portion of which is open to traffic, the Northeast Expressway from the Mystic River Bridge to Revere presently under construction, and a few small portions of Route 28 in the Lawrence-Andover area.

The remaining mileage, exclusive of a portion of the East West Turnpike, is included in the program of work contemplated under the proposed bill.

The design of these highways will pose many perplexing and interesting problems, some relating to engineering, but many more pertaining to Public Relations. Most of you are familiar with some of the public relations problems encountered in locating the Central Artery. Any of you associated with Consultants engaged on our work, know all too well the discouraging delays and frustration which develops from study after study to determine, not only the best line from a traffic or engineering point of view, but from a public relations viewpoint; that is, the best line with particular reference to avoiding damage to business establishments, a dwelling or some "sacred cow."

The Federal Aid Act of 1956 has not made this vexing problem any easier. In fact, hearings are now required to be held, in a con-

venient location, to determine the economic impact of Interstate highways on a community. Massachusetts has always been required to hold hearings prior to laying out a highway on a new location, and Congress has now made this mandatory in other states. We now, however, must hold the hearing in a convenient location, which results in the gathering of a larger number of people who, for one reason or another are opposed to the project. The curious fact is that the holding of the hearing in a convenient location has practically eliminated those who used to appear in favor of a project.

Recently, Professor John T. Howard, Associate in Planning at M.I.T., in a speech in San Francisco made a plea for more co-operation in planning highways between city planners and highway officials. Massachusetts is meeting with responsible city officials and city planners in the development of highway projects that affect the municipality and we have met with some success. We find, however, that due to lack of financing, many city development plans are in a nebulous stage and that the location of the highway being planned, oftentimes becomes the deciding factor in firming up planning on some city development project. Without the highway, the project is dormant. With the highway, oftentimes distorted by city planners to better aid the development at a considerable detriment to traffic service, the development then assumes major proportions. We are continually striving to maintain the integrity of traffic service, while at the same time we want to serve the cities' needs, both actual and future.

As to the design features, it is planned to provide safe, adequate highways for 1975 traffic volumes. The American Association of State Highway Officials prepared geometric design standards for the Interstate system and these were adopted in July of 1956 by the Bureau of Public Roads. As a member of the Design Policy Committee of the American Association, it was my good fortune and privilege to have had a part in forming these standards.

Probably the principal design standard is that one which prescribes full control of access for highways on the Interstate system. Let me read you a portion of the general introduction to the design standards to indicate the importance of this feature and I quote, "The National system of Interstate and Defense Highways is the most important in the United States. It carries more traffic per mile than any comparable National System and includes the roads of

greatest significance to the economic welfare and defense of the nation. The highways of this system must be designed in keeping with their importance as the backbone of the nation's highway systems. To this end, they must be designed with control of access to insure their safety, permanence and utility and with flexibility to provide for possible future expansion." In this connection, I told you that the highways would be designed to handle anticipated 1975 traffic volumes. However, despite the best analysis of present traffic, traffic growth factors, and new development factors, it would appear that as highway engineers, we have not yet been able to raise our sights high enough to provide for future traffic volumes. Most every highway that we build seems to be saturated within a few years after its completion. This however, may be due to the fact that a completed network has not been made available to the traveling public which results in greater attraction to the new Highway despite unfavorable time-distance considerations. The design standards specify that all railroad crossings, grade crossings shall be eliminated for all through traffic lanes and no at grade intersections of other highways will be permitted.

The design speed of all highways in the system shall be at least 70, 60 and 50 miles an hour for flat rolling and mountainous topography and therefore in Massachusetts we are designing our highways for 60 miles an hour. Curvature, super elevation and sight distance are to be correlated with design speeds and for a design speed of 60 miles an hour, the gradients generally shall not be steeper than 4%. In particularly rugged country however, these grades can be increased somewhat.

With respect to the width and number of lanes, the standards specify that traffic lanes shall not be less than 12 feet wide, however, in Massachusetts on our multi-lane highways, we are designing the higher speed lanes for a 13 foot width, with the outer travel lanes 12 feet in width. On divided highways, medians at least 36 feet wide shall be provided, but in urban or mountainous areas, medians can be reduced to 16 feet, and in the city on structures such as our Central Artery, viaducts, or in areas of excessively high right of way costs, medians 4 feet wide are permitted. It is not necessary to provide curbs on all of these medians, however, they may be used where necessary to prevent traffic from crossing the median and in Massachusetts at the interchanges, we are using curbing to prevent cross-

ing and to clearly mark the travel paths; also when narrow medians are employed, the continuous barrier curbs must be offset at least 1 foot from the edge of the through traffic lanes, and where vertical elements more than 12 inches in height, other than abutments, piers, or walls are located in the median, a lateral clearance of 3½ feet from the edge of the through traffic lane to the face of such element is required.

In order to promote safety and to prevent the number of rear end collisions which have been occurring on many of our limited access highways, all weather shoulders not less than 10 feet wide are to be provided for the entire length of the highway and these may only be omitted when a long viaduct or bridge is encountered. To promote safety and to improve the appearance of the highway, slopes, and side slopes are to be 4 to 1 or flatter where feasible but in any case not steeper than 2 to 1.

In order to adequately protect the right of way, to provide for future expansion and to promote greater safety, the right of way width has been increased considerably. For 2 and 4 lane highways, a right of way width of 150 feet is the minimum, for 6 lane divided highways, 175 feet, and for 8 lane highways a 200 feet width is the minimum. In Massachusetts we are presently taking an average of 300 feet for our highways in order that we can provide a median strip wider than the minimum required and we believe that this additional width will provide for further expansion at a later date when traffic volumes require additional travel lanes.

With respect to bridges, all bridges are to be built with a clear height of 14 feet from the pavement to the under side of the bridge and of course an allowance is generally made for resurfacing the highway so that the 14 feet is maintained continuously. Also on bridges less than 150 feet long the all-weather shoulders 10 feet wide are carried completely through the section.

In designing bridges on the Interstate system, the lateral clearance from the edge of the through traffic lane to the face of walls, abutments or piers, shall always be the usual shoulder width, but in any case not less than 8 feet on the right-hand side, nor less than 4½ feet on the left or fast traveled side, and on long span structures or in tunnels where the shoulder is not carried through, safety walks are to be provided in order to protect the motorists who may have suffered a breakdown on the long span or in the tunnel.

What will the development of our Interstate system mean to the Commonwealth? From the engineers' viewpoint, it means a lot of work to spend wisely 100 million dollars per year. Consultants will be used in complex projects to augment the work of our own capable forces. We expect to have a program of planned work soon available which will interest consultants. By planned work, I mean projects in which considerable preliminary planning has been accomplished so that the consultant will be in a position to develop design details, and construction cost for preliminary review on a line known to be the best feasible route. Little time is then lost proceeding to the preparation of final construction plans, estimates, and specifications.

From the motorists' point of view, it means a considerable savings in operating costs. It has been estimated that travel on limited access highways can result in a savings of between 1 and 2 cents a mile for the average passenger vehicle operator.

For truckers, even greater savings in operating cost can be realized, since it has been estimated by the American Truckers Association that savings in truck operation will amount to about 5 cents per vehicle mile and in driving time alone it is estimated that the trucking industry can save about 900 million dollars per year.

Also from the motorists' and truckers' point of view, there can be a great advantage in the present cost of accidents.

Under our highway planning studies, the Commonwealth of Massachusetts has been co-operating with the Bureau of Public Roads in a study of direct accident costs. To date, we are the only state that has entered into such a study and we were selected because of the fact that our compulsory insurance laws made it necessary for us to keep detailed reports on all accidents. The work of preparing the report therefore, is the joint effort of the Massachusetts Registry of Motor Vehicles, the Department of Public Works and the Bureau of Public Roads.

The first portion of the study has been completed, which dealt with the cost of accidents to passenger cars and since the study was started in 1954, the accident records of 1953 were used.

Our study showed that in 1953 a total of 214,678 accidents to passenger cars occurred on the highways in Massachusetts and that the total cost of these accidents amounted to $50,223,500. That cost is the direct cost, that is, it is composed of money values of damage to property, hospitalization, doctors, dentists, nursing service, ambu-

lance use, medicine, damages awarded in excess of other direct costs, attorney services, court fees, etc. Indirect costs which are composed of the money value of such things as loss of future earnings, the overhead costs of accident insurance, high school driver training, safety engineering, traffic courts, etc., will not be available for some time, because in the third and final part of our study we will attempt to determine the indirect cost of motor vehicle accidents to all citizens of Massachusetts. It is interesting to note however, that with a population of 5 million people, our accident cost amounts to 50 million dollars, or a total of $10.00 for every man, woman, and child of this state.

With respect to the type of accidents, you may be interested to know that 83% or $41,800,000 of the total direct cost of passenger car accidents are the result of the collision of vehicles; and angle collision costs of 17 million dollars was the biggest single item. Our studies also showed that the total cost of rear-end collisions was more than that of head-on collisions, and the total cost of accidents involving pedestrians was more than twice that of non-collision accidents.

Studies have shown that the accident rate on limited access Expressways is only 1/3 of that which prevails on primary highways where railroad grade crossings exist, and where streets are permitted to intersect at grade. We are confident therefore, that the savings in accident costs alone will be substantial and would more than pay for the improvements to the Interstate Highway System.

In addition however, every citizen of the state will benefit, since the cost of transporting produce and merchandise transported by truck can be reduced through more efficient travel on limited access highways, thus reducing ultimate consumer costs.

Cities and towns can benefit through increased real estate evaluations, despite the first impact of loss on present assessed values due to the highway location or to highway takings.

Route 128 is known nationally for the industrial development which it has generated along its borders. To a large extent, this pattern will, we expect, be repeated along other major routes.

We have a gigantic task before us. Engineers, who have always played an important role in the development of our state and nation, once again have an opportunity to contribute to the economic improvement of their state, by wisely planning better transportation facilities, by making the fullest use of available engineering talent,

by taking advantage of proving that new computation methods, involving the use of electronic computers, by greater use of aerial survey and photogrammetric plans and through research, by developing new time saving methods, we can do this job and when it's done there'll be more highway work in the future. Remember, we have just discussed the Interstate System and remember that the needs for other systems of highways still remain to be met.

OF GENERAL INTEREST

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETING
Boston Society of Civil Engineers

APRIL 15, 1957.—A Joint Meeting of the Mass. Section, American Society of Civil Engineers and the Boston Society of Civil Engineers was held this date at the Faculty Club, Mass. Institute of Technology and was called to order by President A. Russell Barnes of the ASCE at 7:50 P.M., following a dinner preceding the meeting.

President Barnes called upon President Casagrande of the Boston Society of Civil Engineers to conduct any necessary business.

President Casagrande requested the Secretary to present recommendation of the Board of Government to the Society for action. The President stated that this matter was before the Society in accordance with provisions of the By-Laws and notice of such action published in the ESNE Journal dated April 8, 1957.

The Secretary presented the following recommendation of the Board of Government to the Society for initial action to be taken at this meeting.

> *MOTION* "that the dues of Resident Members be increased from $10 to $15 per year, retroactive to March 20, 1957, and that the dues for all other categories of membership remain the same as at present".

On motion duly made and seconded it was *VOTED* "that the dues of Resident Members be increased from $10 to $15 per year, retroactive to March 20, 1957, and that the dues for all other categories of membership remain the same as at present".

President Casagrande stated that final action on this matter would be taken at the May 15, 1957 meeting of the Society.

The Secretary announced the names of applicants for membership in the BSCE, and that the following had been elected to membership on April 8, 1957:

> *Grade of Member*—Robert J. Basso, John H. Cullinan, Howard J. Farquharson,* Robert J. Frazier, Joseph A. Grimaldi,* Thomas C. McMahon, Kenneth F. Mercier,* Charles A. Parthum, Reginald L. Reed, Louis J. Richard, Nicholas R. Samaha, Frank E. Trotto,* Leonard W. Tucker.
>
> *Grade of Student*—Robert H. Stewart, Maurice C. Vezina.

President Barnes then turned the meeting over to Dr. John B. Wilbur, who acted as Moderator in the panel discussion "Why Two Engineering Societies in Boston". The panel members were E. B. Cobb, A. Haertlein, J. G. W. Thomas and E. Ireland.

Seventy one attended the dinner and eighty seven attended the meeting.

The Meeting adjourned at 9:55 P.M.

ROBERT W. MOIR, *Secretary*

MAY 15, 1957.—A regular meeting of the Boston Society of Civil Engi-

*Transfer from Grade of Junior.

neers was held this date at the Hotel Lenox, 61 Exeter Street, Boston, Mass., and was called to order by President Arthur Casagrande, at 7:15 P.M.

President Casagrande stated that the minutes of the previous meeting held on April 15, 1957 would be published in a forthcoming issue of the JOURNAL and that the reading of the minutes of that meeting would be waived unless there was objection.

The Secretary announced the names of applicants for membership in the BSCE, and that the following had been elected to membership on May 13, 1957.

Grade of Member—John Podger, Daniel F. Tully.

Grade of Associate—George B. Kehoe.

There being less than the requisite 25 members (needed for a quorum) present, no action could be taken on the recommendation of the Board to increase the dues of Resident Members to $15.

President Casagrande introduced the speakers of the evening, John B. McAleer, Project Engineer, and Peter J. A. Scott, Project Engineer, N. E. Div. Corps of Engineers, Boston, Mass., who presented a paper on "Hurricane Protection in New England", to a small but enthusiastic audience.

A lively discussion followed after which a rising vote of thanks was given the speakers.

The meeting adjourned at 9:15 P.M.

ROBERT W. MOIR, *Secretary*

JUNE 3, 1957.—A Special Meeting of the Boston Society of Civil Engineers was held this date at the Society Rooms, 715 Tremont Temple, Boston, Mass., and was called to order by Vice-President William L. Hyland at 12:30 Noon.

Vice-President Hyland stated that the minutes of the previous meeting held May 15, 1957 would be published in a forthcoming issue of the JOURNAL and that the reading of the minutes of that meeting would be waived unless there was objection.

Vice-President Hyland stated that at the April 15, 1957 meeting of the Society the Secretary presented a recommendation of the Board of Government which had been acted upon favorably and that final action on the recommendation would now be taken. Notice of such action published in the ESNE Journal dated May 27, 1957.

The Secretary read the following motion:

MOTION "that the dues of Resident Members be increased from $10 to $15 per year, retroactive to March 20, 1957, and that the dues for all other categories of membership remain the same as at present".

On motion duly made and seconded it was *VOTED* "that the dues of Resident Members be increased from $10 to $15 per year, retroactive to March 20, 1957, and that the dues for all other categories of membership remain the same as at present".

Vice-President Hyland stated that this was the final action on this matter.

The Secretary announced the names of applicants for Membership in the BSCE.

Forty one members attended the meeting.

The meeting adjourned at 12:40 P.M.

Respectfully submitted.

ROBERT W. MOIR, *Secretary*

HYDRAULICS SECTION

MAY 11, 1957.—A meeting of the Hydraulics Section was held at the Alden Hydraulic Laboratory of the Worcester Polytechnic Institute, Holden, Massachusetts, beginning at 10:00 A.M.

Professor Leslie J. Hooper, Director of the Laboratory, discussed the history of the Laboratory, its present operation and the possibility that future

expansion will require the addition of pumps to the hitherto adequate gravity water supply. Professor Hooper and his assistant, Professor Lawrence C. Neale, an officer of this Section, then conducted a tour of the Laboratory where the facilities were explained and the operation of scale models including dam spillways and a river intake were observed and discussed.

The tour ended at noon at which time Mr. Clyde W. Hubbard, Chairman, expressed the gratitude of the Section for the hospitality of the Laboratory. The meeting then informally adjourned. Total attendance was 35, including members of the Laboratory staff and wives and children of guests and members.

LEE MARC G. WOLMAN, *Clerk*

ADDITIONS

Members

Clifford E. Adams, 5 Briarwood Trail, Weymouth, Mass.
Robert J. Basso, 48 Spring Street, Melrose 76, Mass.
Gino Cosimini, 96 E. Gaffey Hgts., Fort Knox, Kentucky
John H. Cullinan, 16 Elwin Road, Natick, Mass.
Howard J. Farguharson, 19 Fiske Street, Waltham 54, Mass.
Walter R. Ferris, Pierce Hall, Harvard University, Cambridge, Mass.
Alfred F. Ferullo, 10 Birch Street, Milton, Mass.
Robert J. Frazier, 8 Centennial Avenue, Gloucester, Mass.
Allison C. Hayes, 172 Ashland Street, Melrose 76, Mass.
Charles D. Hopkins, River Forecast Center, Bradley Field, Windsor Locks, Conn.
Ronald C. Hirschfeld, 46 Highland Avenue, Cambridge, Mass.
James H. Kane, 56 Woodard Road, W. Roxbury 32, Mass.
George B. Kehoe, 52 Ridge Road, Milton, Mass.
John C. Matte, 119 Pond Street, So. Weymouth, Mass.
Thomas C. McMahan, 121 Thornton Road, Waltham, Mass.
Charles A. Parthum, 33 Russell Street, Marblehead, Mass.
Robert F. Pelletier, 44 Salem Street, Wakefield, Mass.
John Podger, 31 Highland Road, Wellesley, Mass.
Arthur Quaglieri, 12 Hillcrest Road, Norwood, Mass.
Louis J. Richard, 22 Talbot Avenue, No. Billerica, Mass.
Nicholas Samaha, 1179 Liberty Street, So. Braintree, Mass.
Frank E. Trotto, 108 Shawsheen Avenue, Wilmington, Mass.
Leonard W. Tucker, 32 Fletcher Road, Belmont 78, Mass.

Juniors

John W. Cossart, 24 Hamilton Avenue, No. Quincy, Mass.
Daniel F. Tully, Newmarket, New Hampshire.
James R. McElvenny, 93 Mora Street, Dorchester, Mass.

DEATHS

Horace L. Clark, March 3, 1957.
James M. McNulty, May 11, 1956.
Robert W. Pond, February 17, 1957.

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The advertising pages of the JOURNAL aim to acquaint readers with Professional and Contracting Services and Sources of Various Supplies and Materials. You would find it of advantage to be represented here.

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