**Zeitschrift:** IABSE congress report = Rapport du congrès AIPC = IVBH Kongressbericht

**Band:** 2 (1936)

**Artikel:** Soil studies for the Storstrøm bridge, Denmark

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**DOI:** https://doi.org/10.5169/seals-3234

# VIII 1

# Soil Studies for the Storstrøm Bridge, Denmark.

## Bodenuntersuchungen für den Bau der Storstrøm-Brücke in Dänemark.

## L'auscultation du terrain pour la construction du pont Storstrøm, Danemark.

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*Introduction.*

In this paper are to be mentioned the soil studies carried out in connection with the construction of the Storstrømsbridge, Denmark, built in the years 1933—37 for the Danish State Railways.

The substructure now nearly completed has been carried out by Messrs. Christiani & Nielsen, Copenhagen. Messrs. Dorman, Long & Co., Middlesbrough, England, have carried out the superstructure.

The Storstrømsbridge will carry a single track railway, an automobile road and a foot-path over the Sound "Storstrømmen", thus providing a connection between the islands of Zealand and Falster.

The bridge will be abt. 3200 m long and have 51 piers. In "Storstrømmen" the average depth of water is abt. 8 m; the piers are generally founded at a depth of 2—3 m below bottom, directly on clay. Maximum foundation depth at level $\div$ 16 m.

The soil consists of glacial clay of varying consistency, under which the chalk is found in greater depths as seen on the longitudinal section. Fig. 1.

On account of the great number of piers and the relatively small variations in the depth of water in the bridgeline it has been possible to standardize the methods of construction for the greater part of the piers.

Where the clay on which the piers are founded, is of sufficient resistance. the pit was pumped dry, and the foundation slab and the pier shaft concreted in the ordinary way. Where the bottom was weaker the foundation slab was concreted under water, whereupon the water was pumped out, and the pier shaft concreted in the dry.

Only the first method of construction will here be discussed in details, as it imposed the greatest stresses on the clay of the bottom and on the steel sheet piling which served as a cofferdam for the lower part of the pit. Further this method has been of special interest on account of the rapid execution of the work (one pier generally completed up to level + 3 m in less than a month) and the possibility of measuring during the execution of the work the stresses set up in the steel sheet piling.

This method has heretofore been employed for 24 piers and without exception with satisfactory results.

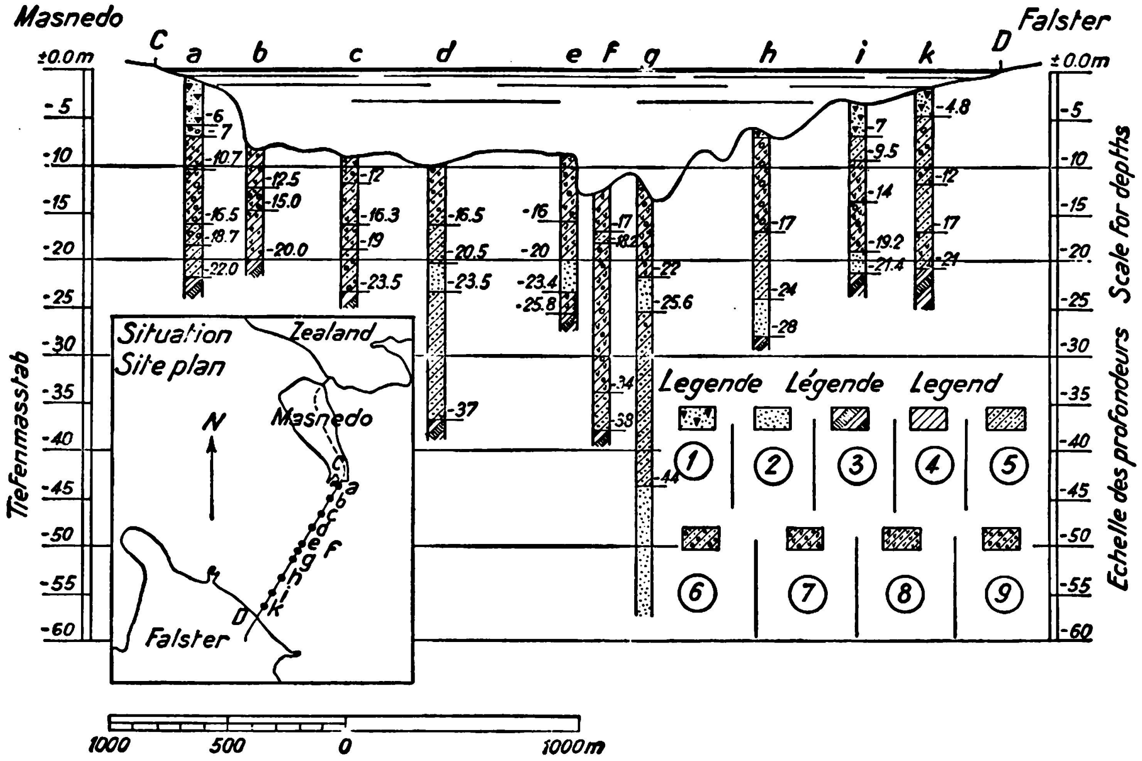


Fig. 1.

Geological Section.
1) mud.
2) sand and gravel.
3) chalk.
4) clay.
5) clay w. sand.
6) clay with sand and stones.
7) clay with limestone and stones.
8) clay with limestone.
9) clay with stones.

Fig. 2 shows how a floating ring-shaped steel cofferdam, a so-called unit, is used for the upper part of the pit. The unit has been towed to the site of the pier in question and lowered by means of water ballast on short wooden piles driven in advance along the contour of the unit.

The outside wall of the unit corresponds to the elliptical contour of the foundation slab. A steel sheet piling has been hung in advance along the outer circumference of the unit, and this sheet piling is driven under water by means of a Mac Kiernan Terry hammer till the upper end of it is just above the lower edge of the unit. The wedge shaped joint between the outside of the unit and the steel sheet piling is tightened by means of a continuous hemp rope, water is pumped out, the joint automatically tightening itself, sometimes helped by divers, and the bottom is laid dry so that excavation of the foundation can take place. During excavation one had the opportunity to check the stresses in the steel sheet piling and the resistance of the clay in the bottom of the pit.

No bracing at all was employed. The unit itself was designed to take the entire water pressure on its outside and also the reaction from the top of the steel sheet piling.

The lower end of the piling was solely supported on the clay bottom.

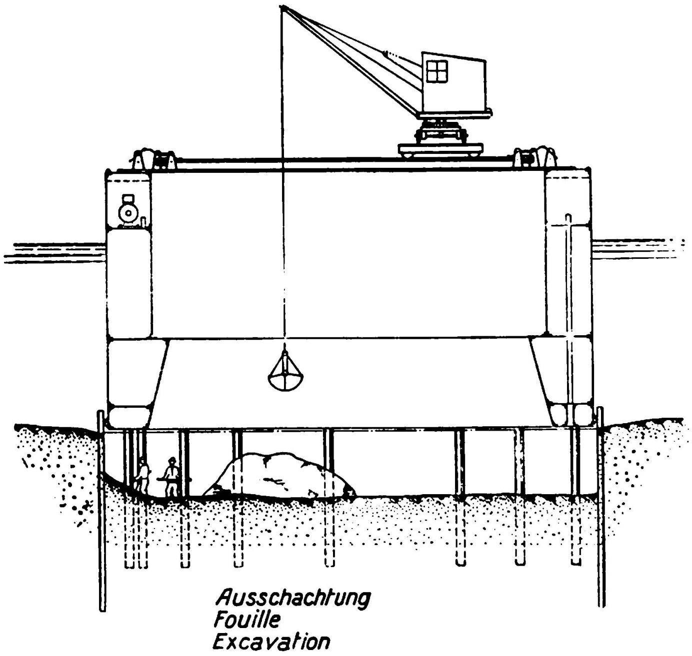


Fig. 2.

Foundation by use of Unit.

When the excavation was finished the foundation slab was concreted against the steel sheet piling, which remained in the pier as a protection against scour.

The inside wall of the unit served as a form for the bottom part of the pier shaft, which was concreted only up to 3 m below water level. When the concrete had hardened the unit was removed and re-employed for another pier of the same type.

That part of the pier shaft which was between 3 m below water and 3 m above water was constructed on a slipway as a reinforced concrete caisson, without bottom, covered with granite ashlar on the outside.

This caisson was lowered in the water by a carriage running on the slipway and suspended between two barges and thus towed to the pier site and lowered in exact position. Previously an asphaltic tightening material had been placed on the top of the pier at level + 3 m; the water could immediately thereafter be pumped out and the caisson filled with concrete.

The upper part of the pier shaft was concreted in the normal way inside steel forms.

The statical conditions of the sheet piling were as described above extraordinarily clear, especially as the deflection of the sheeting caused an open joint to be formed on the outside between the piling and the clay, so that the water pressure must at the time of failure be reckoned to act right down to the bottom end of the sheeting.

This condition was considered at the breaking limit of the structure, and the steel sheet piling was designed to resist this water pressure, including a certain margin to allow for high water conditions, at stresses near the yield point of the steel.

The resistance of the clay could of course not be ascertained with so much accuracy as that of the steel sheet piling and a factor of safety of abt. 1,5, therefore, was introduced in the values found for the shearing strength of the clay on the basis of laboratory tests with the cone-apparatus described below.

*Borings and Sampling Operations.*

Before the type of foundation was decided upon and the steel sheet piling was designed more than a hundred borings were made at the site of the piers and several samples of the soil were taken at each boring for testing in the laboratory.

The boring operations themselves were executed in the ordinary manner as wash borings from a large barge carrying a set of boring apparatus at each end, so that two borings were made at the same time.

A detailed record of all the boring operations were made, stating the boring velocity under different conditions, exact depth of samples taken etc.

In order to get the samples, which were taken at short intervals, as undisturbed as possible a special hydraulic clay sampler was designed. Fig. 4 shows this clay sampler in details, while the general arrangement for the sampling operations is seen on Fig. 3.

The clay sampler is designed to take samples of abt 48 mm diameter.

It consists of a steel cutting tube lined with a thin brass tube in which the sample is introduced during the operations. This cutting tube is connected to a piston moving in a cylinder of abt. 76 mm inside diameter. In the centre of the cylinder is a fixed guiding rod ending in a bottom plug, which acts as a piston in the brass tube, when this tube, together with the steel cutting tube and the main piston, is moved downwards in the cylinder by means of water pressure acting on the upper side of the main piston. The water under the main piston will escape through the centre bore in the guiding rod.

To operate the sampler it is connected to pressure pipes as shown on Fig. 5, screwed tight together by means of conical threads, and lowered in the casing tube seen on Fig. 3, with piston locked in its top position and the lower end of the cutting tube flush with the bottom plug, until this reaches the bottom. Soft material left on the bottom is thus displaced.

Then a clamping piece is bolted on to the pressure pipe and connected to the loading beams bolted to the casing tube. These beams are loaded by sand boxes or pig iron supplying the reaction to the pressure needed for the cutting out of the sample.

Now water from the pressure pump is led through the pipe into the cylinder and causes the piston to move so that the cutting tube will penetrate into the ground and cut out a sample of clay.

When the piston reaches its bottom position, which is observed on the manometer of the pump by a sudden increase in the pressure, the piston is automati-

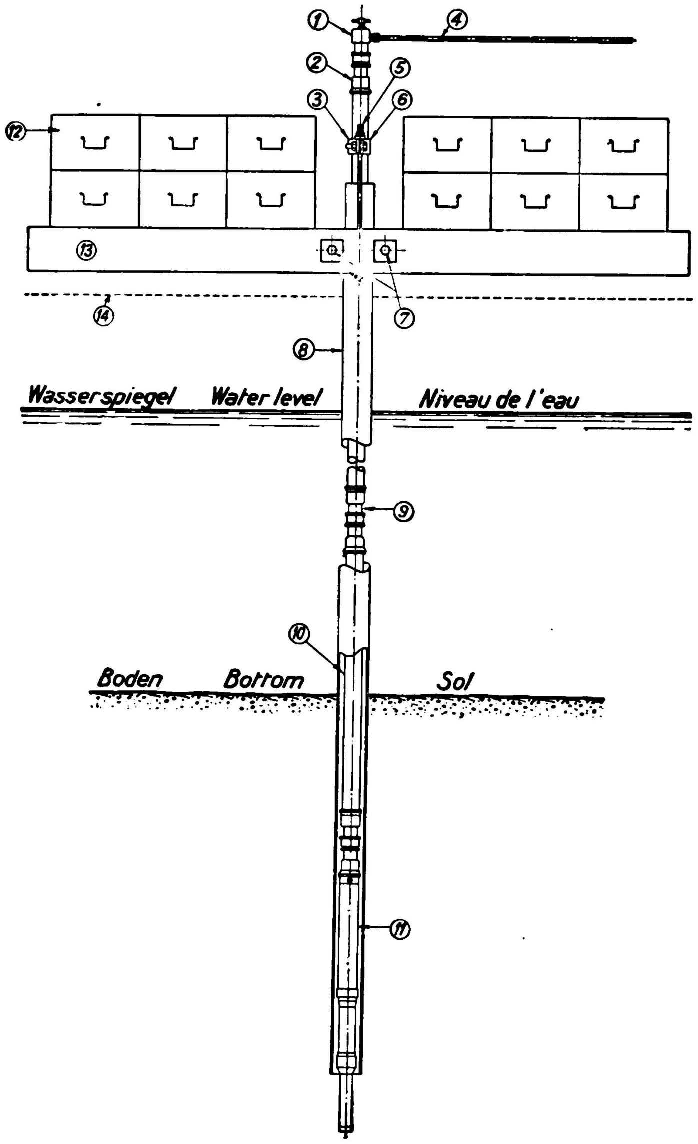


Fig. 3.

Sampling operation with hydraulic clay sampler.

1) Cap with air valve.
2) socket.
3) clamping piece.
4) from force pump.
5) 2 — 5/8" bolts.
6) 2 — 3/4" bolts.
7) 2 — 1" bolts.
8) casing pipe.
9) coupling.
10) hydraulic pressure pipe.
11) hydraulic clay sampler.
12) sand boxes (or pig iron).
13) loading beams.
14) floor level of barge.

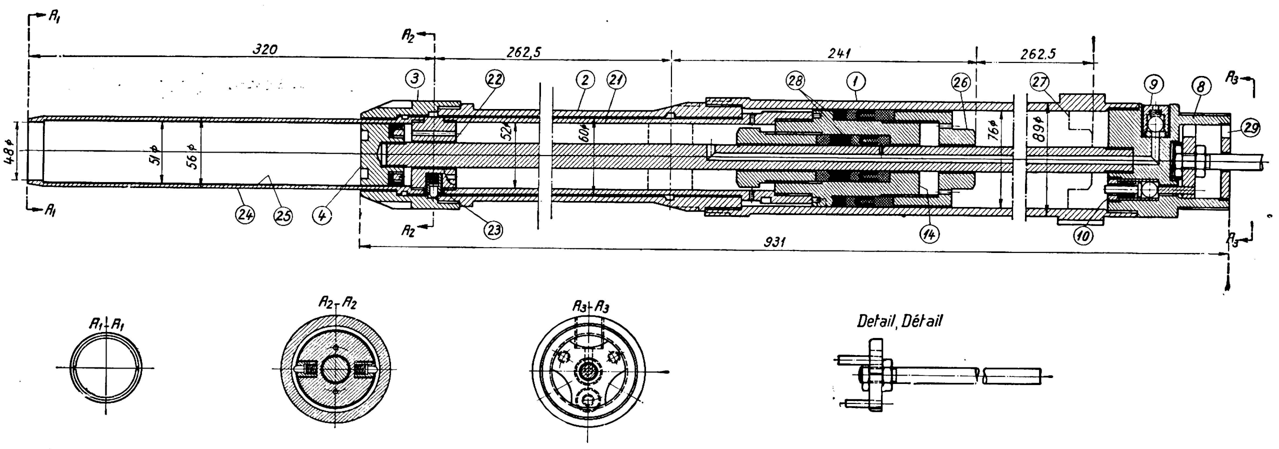


Fig. 4.

Hydraulic clay sampler.

Hydraulic clay sampler, Detail.

Note: Piston shown in bottom position.

Explanations:

1) Main cylinder.
2) Cylinder extension.
3) Screw cap.
4) Central tightening plug.
8) Cylinder head.
9) Escape valve.
10) Non-return valve.
14) Piston top.
21) Force piston body.
22) Piston bottom.
23) Spring loaded lockingbolts.
24) Clay sample cutting tube.
25) Detachable clay sample container.
26) Hexagonal head.
27) Top position of piston.
28) Hemp packings.
29) Entrance for pressure water.

cally locked in its position and the sampler is lifted to the deck of the barge, the screw cap is removed and the tube containing the sample detached.

The consistency of the sample is determined immediately on the site by means of a simple cone-apparatus called spring-scale-cone, whereafter the brass-tube is sealed with caps and adhesive tape at both ends and sent to the laboratory for further testing.

The clay sampler is designed for a normal working pressure of max. 50 atm. corresponding to a total pressure on the cutting tube of abt. 2 ts.

It was found that the samples taken by this device were considerably less disturbed than samples taken by instruments of more simple design, intended to be driven into the ground by blows with a hammer.

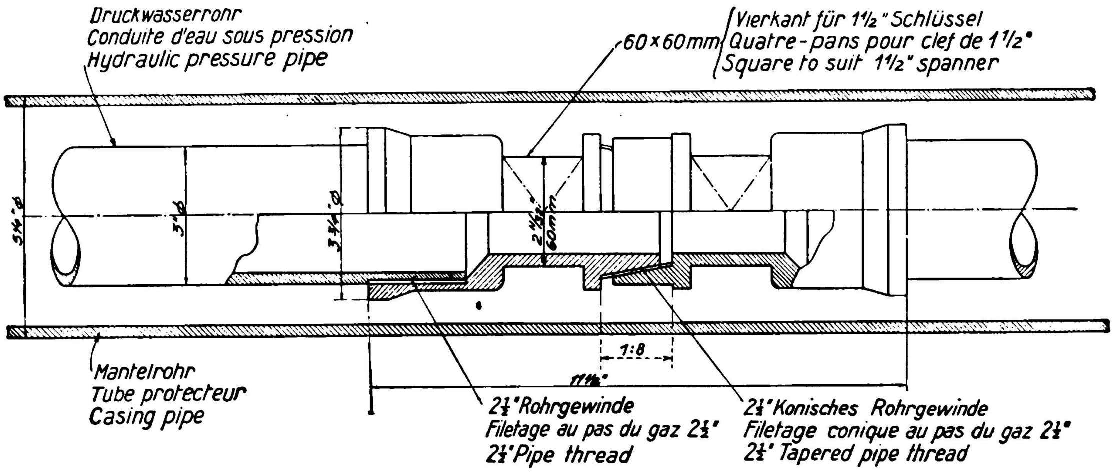


Fig. 5.

Joint in hydraulic pressure pipe.

*Laboratory Test.*

The samples were tested by the author at the Structural Research Laboratory, Royal Technical College, Copenhagen.

In the laboratory were made the ordinary tests of water content, plasticity, shrinkage and liquid limit, specific gravity, granulometric composition, compressibility, permeability, and resistance of cylinders. An apparatus for direct shearing test was not available, but some direct shearing tests were made for a check at the "Laboratorium für Schiffbau und Wasserbau" in Berlin.

The main tests of the consistence of the samples were made by a cone-apparatus devised by the author in 1930, as will be described in detail below.

Further all samples were examined geologically by Mrs. E. L. Mertz of the Danish Geological Survey.

The clay at Storstrømmen is a glacial boulder clay with a considerable content of chalk, as much as abt. 50%.

At greater depths is often found diluvial clay and diluvial sand with varying content of clay. These diluvial strata are less resistant than the boulder clay and are characterized by the extreme sensibility against disturbance.

By remolding of the diluvial clay the resistance generally will be reduced to 15—25% of the resistance of the undisturbed sample.

The water content of the boulder clay ranges from 10—15% of the weight of the dry substance and for the diluvial clay from 18—26%.

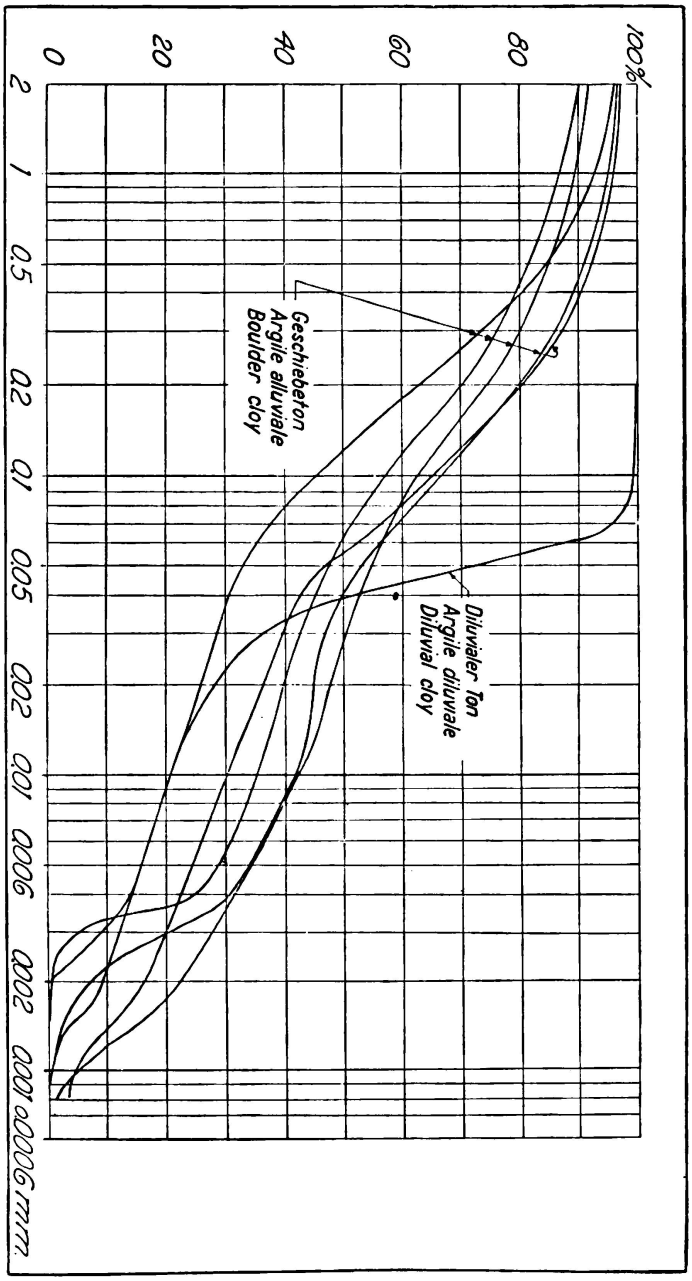


Fig. 6.
Some typical grain-size curves.

For boulder clay the shrinkage limit is found at 8—10%,
the plasticity limit at 10—13%, and
the liquid limit at 20—22%.

Characteristic curves of granulometric composition are shown on Fig. 6.

Also a number of compression tests were made. One example is shown on Fig. 7.

On account of the small water content of the boulder clay on which most of the piers are founded, the settlements of the piers were expected to be very small, which has been confirmed by levelling of some of the piers.

The cone-apparatus employed for testing the consistency of the clay is shown on Figs. 8 and 9.

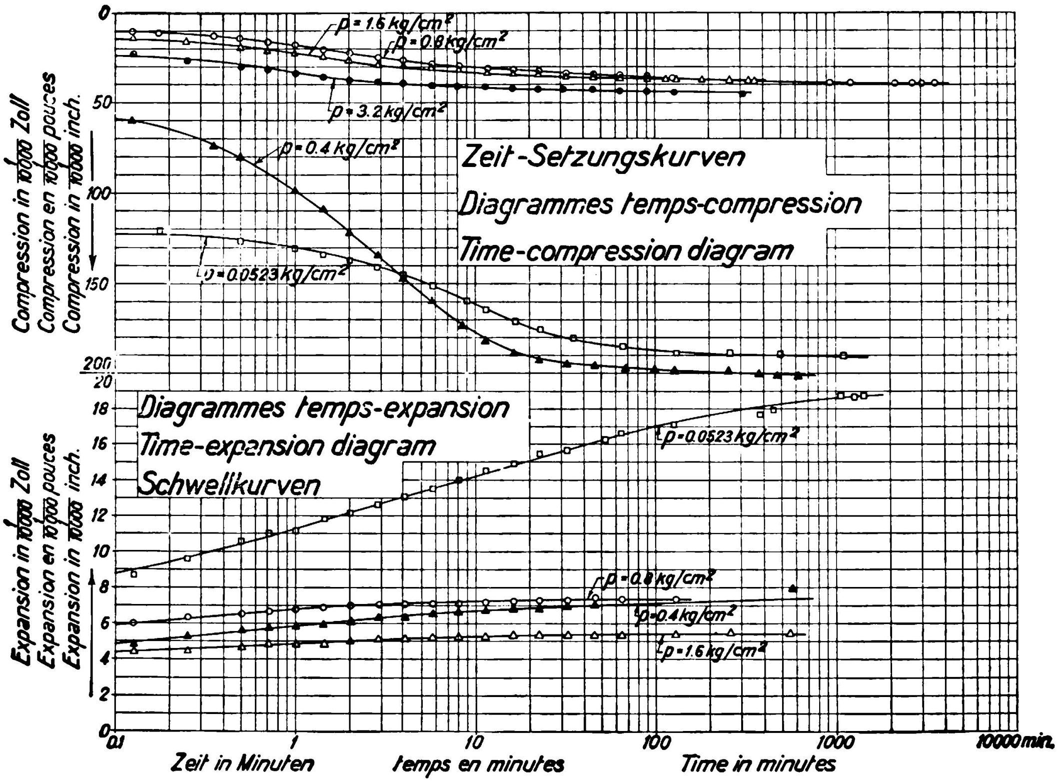


Fig. 7.

Time-compression and -expansion diagrams.

The $60^0$ cone fixed on a vertical rod is loaded with various acts of weights, from 0,3 kilos to 12 kilos, and screwed down until the point just touches the surface of the clay.

The brass tube containing the sample is cut with a saw, so that the sample during the testing operation is surrounded by the brass-ring.

The micrometer screw on the top end of the apparatus is adjusted, the cone with its weight released and the penetration measured by the screw.

The test is made with different weights on the cone and several readings for each weight. The average penetrations y in mm are plotted against the weight G on double logarithmic paper, the weights as abscissas and the penetrations as ordinates, as shown on Fig. 10. The results closely approximate a straight line, and for the same type of clay the inclination of this line against the axis is found to be nearly constant.

The consistency K of the clay is defined as the weight of the cone which gives a penetration of 10 mm.

The results follow the law

$$G = K \cdot \left(\frac{y}{10}^{n}\right)$$

For the boulder clay of Storstrømmen n is on an average = 1,75. The consistency K can be expressed as

$$K = G \cdot \left(\frac{10}{y}\right)^{1,75}.$$

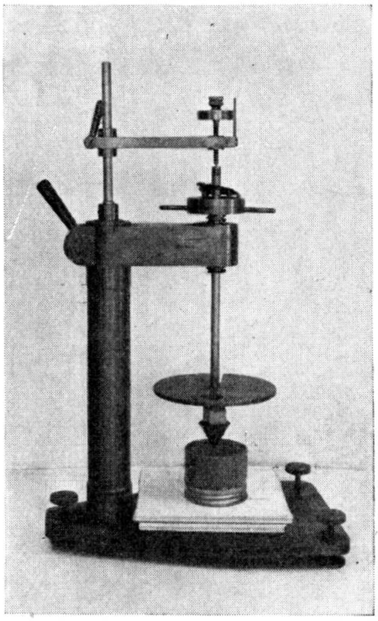

Fig. 8.
Cone test apparatus (Photo).

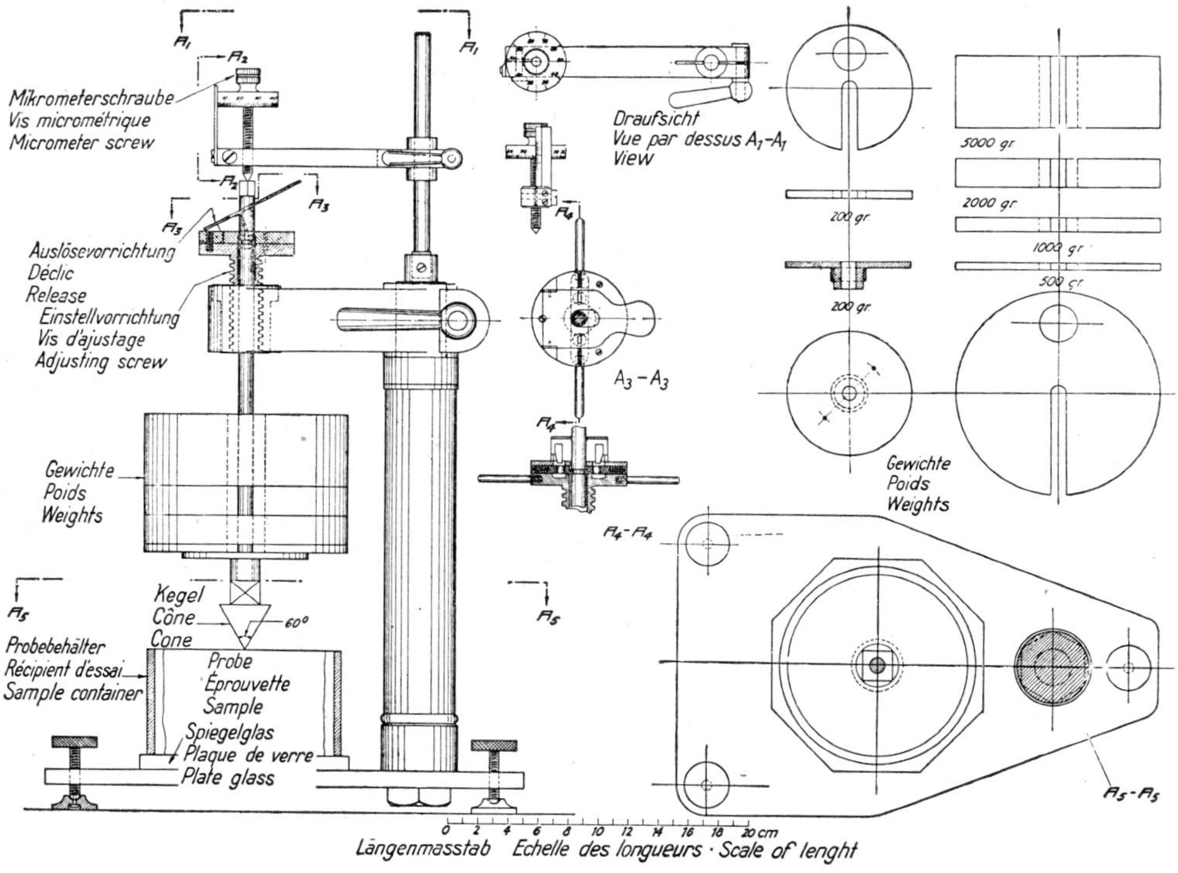


Fig. 9.
Cone test apparatus. Details.

The advantage of this apparatus is that the great weights give penetrations easy to measure with the necessary accuracy, and that errors caused by the existence of small stones in the samples are easily eliminated, when a greater number of observations is taken.

The observations were plotted as shown in a few examples on Fig. 11.

Tests were made with undisturbed clay as well as with remolded clay.

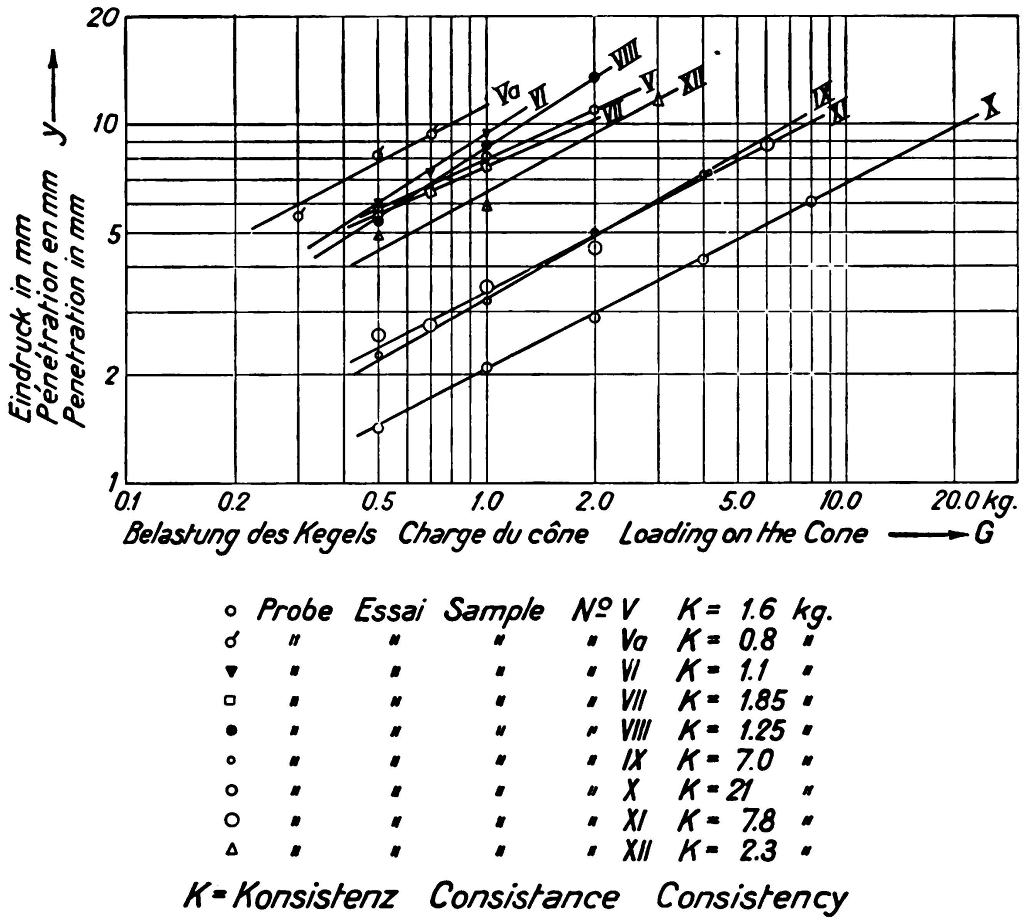


Fig. 10.

Results from tests with cone apparatus.

On the diagrams are further shown the results obtained with the spring-scale-cone on the site. This apparatus is designed by Mr. O. Godskesen, C. E. By comparing the results obtained by the spring-scale-cone with those obtained by the above mentioned cone-apparatus, it was found that the spring-scale-cone on an average gave consistencies abt. 40% higher.

It seems that the samples tested immediately after the extraction had a higher consistency than that found later on by the same method in the laboratory, probably on account of internal swelling.

During the practical execution of the piers the author was, however, not under the impression that the results found in the laboratory were too unfavourable such as was supposed at the start.

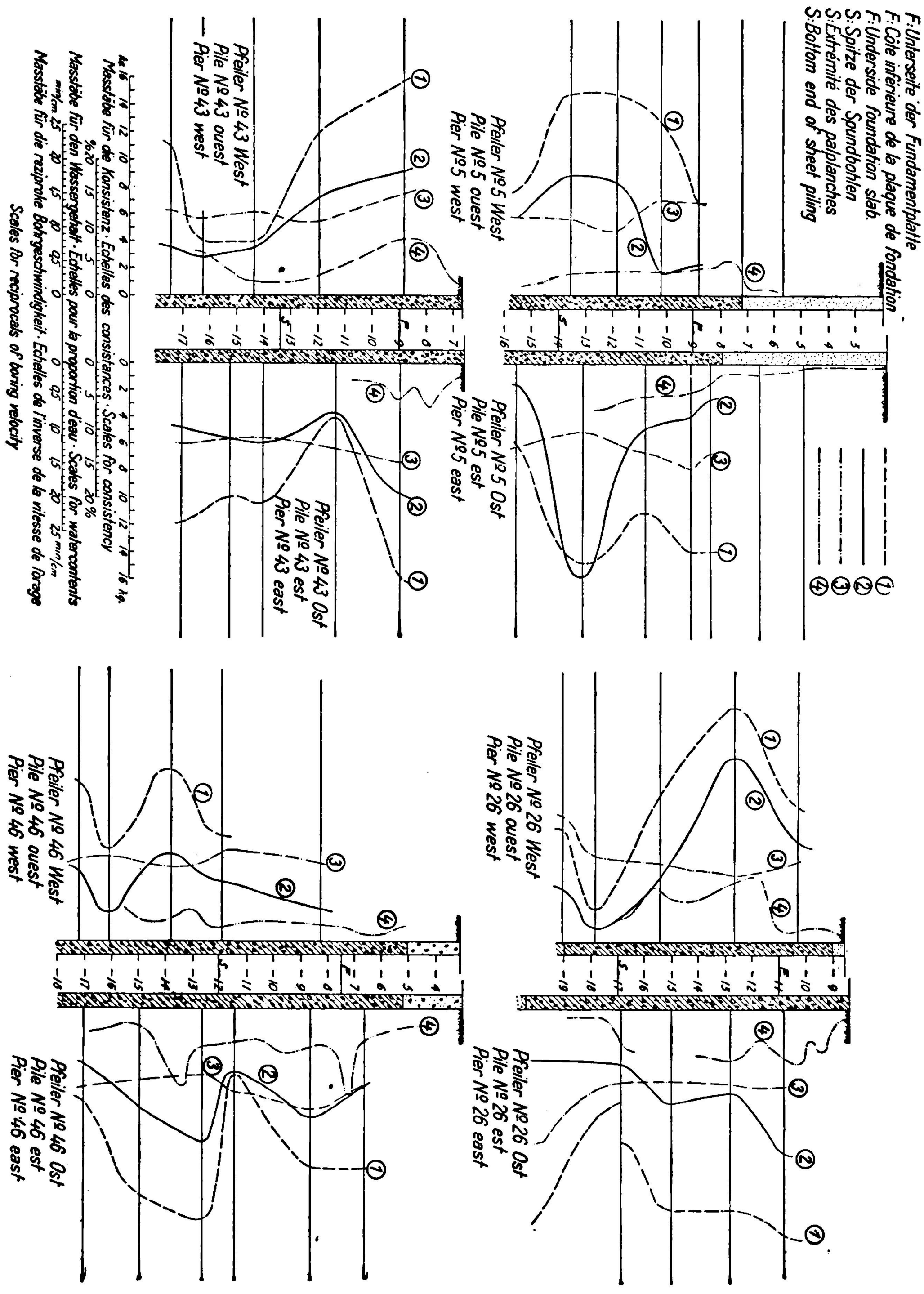


Fig. 11.

Typical consistency diagrams.

1) Consistency determined at the site (Spring scale cone).
2) Consistency determined in the laboratory.
3) Watercontent in per cent of dry substance.
4) Reciprocals of boring velocity.

It therefore seems that a similar swelling must have taken place in the clay of the bottom, when the pit was pumped out and the clay relieved of the water pressure. It appears that the speed of the work was of importance, and that the resistance of the clay decreased when the pit by some reason or other was kept dry longer than usual.

A series of comparative tests was made in the laboratory to determine the relation between the cone-consistency and the cylinder-resistance. The number of these tests can hardly be considered sufficient for this purpose, and further investigations are considered desirable.

The cylinder tests were, however, carried out with a slow increase of the load, so that it must be supposed that the internal friction has been more or less effective.

In the structure, where the pit was kept dry only for a short period, this can hardly have been the case, and it was assumed by the calculations that only the cohesion of the clay determined its resistance, and that internal friction could be neglected.

According to the above mentioned cylinder tests the breaking strength d in kilos/cm$^2$ of the cylinders was found on an average to be about:

$d = 0.5 \cdot K$ (K consistency in kilos).

Neglecting the friction, the shearing strength of the clay ought to be

$e = 0.5 \cdot d$

and consequently

$c = 0.25 \cdot K$.

Practical experience showed, however, that this value of the shearing strength was too high. This can be explained partly by the slow speed of the cylinder tests, and partly by the fact that the clay in the pit probably will be more or less disturbed during the pumping out of the pit.

The deformations of the steel sheet piling were considerable, so that at least the upper part of the clay supporting the wall was disturbed during the pumping out.

Tests with completely remolded samples showed that the consistency was reduced to abt. 45% of the consistency of undisturbed samples.

By the practical calculations the shearing strength of the clay was therefore only taken at

$c = 0.1 \cdot K$,

which value was proved by the check measurements of the sheet piling to be fairly close to the actual resistance.

In the opinion of the author the cone test employed on impermeable clay will express the cohesion of the clay. On account of the great speed of the test the effect of internal friction can be neglected, so that results agreeing with the shearing strength observed in actual practice will be obtained.

The ratio between cohesion and consistency can, however, not be considered constant, but must be determined for each special type of clay.

*Statical Calculation of steel sheet piling.*

A simple method of calculation was sought for determining the necessary driving depth of the steel sheet piling under the foundation level, so that the shearing resistance of the clay was not exceeded, and for finding the corresponding bending moments in the sheet piles.

According to what is mentioned in the introduction the dimensioning was done under the assumption that the sheet piling was curved inwards just to the bottom end, and that the full water pressure, including a margin of 1 m for high water conditions, was acting from the outside in the full height (fig. 12a).

Under these circumstances the limit of the resistance of the clay must be supposed to be reached, and the bending moments in the sheeting to be a maximum. In the calculations were introduced a shearing resistance (cohesion) c = 2/3 · 0.1 K; a safety factor for the clay of 1.5 was thus intended.

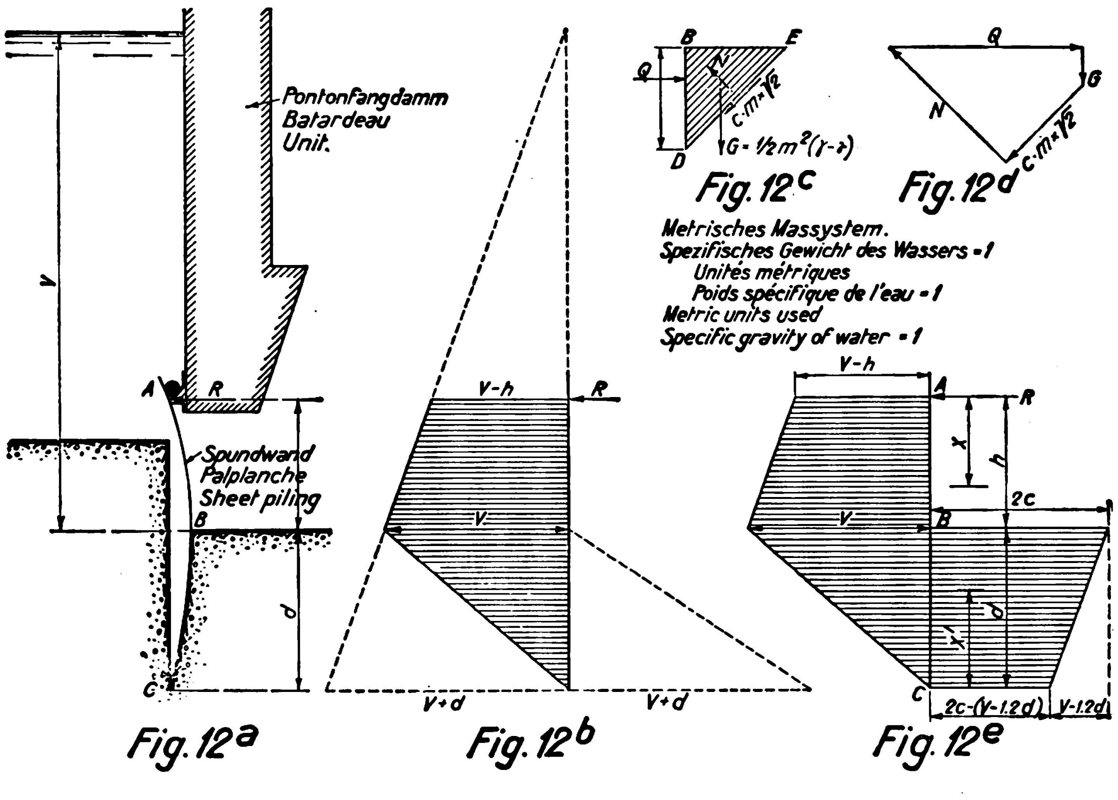


Fig. 12.

Calculation of steel sheet piling.

Inside the sheeting, the water pressure in the voids of the clay must decrease from the full value existing at the bottom end on the outside to zero at the foundation level. For the sake of simplicity this variation is taken according to a straight line, as indicated in Fig. 12b, which shows the pressure exerted on both sides of the wall, and the difference between them.

The gradient of pressure will be:

$$\alpha = \frac{v + d}{d} = 1 + \frac{v}{d}$$

i. e. the interior buoyancy in the clay is equal to the ordinary statical buoyancy multiplied by $\left(1 + \frac{v}{d}\right)$.

Under the resultant water pressure the sheeting will press against the clay. Sliding is supposed to take place along planes of rupture under 45°. Considering a sliding wedge, B D E in Fig. 12 c, this is seen to be subjected to the following external forces: The horizontal reaction Q from the sheeting; a vertical force G = weight of wedge minus buoyancy; the cohesion $c\,m\sqrt{2}$ acting along the plane of rupture DE and an unknown reaction N perpendicular to the same plane DE.

As the specific gravity $\gamma$ of the clay in the actual state of moisture can be taken as 2.2 (specific gravity of water = 1), it is found that

$$G = \frac{m^2}{2}\left[2.2 - \left(1 + \frac{v}{d}\right)\right] = m^2\left(0.6 - \frac{v}{2d}\right).$$

Fig. 12 d shows the force polygon and by projection on a line parallel to the plane of rupture, it is seen that

$$\frac{Q}{\sqrt{2}} = \frac{m^2}{\sqrt{2}}\left(0.6 - \frac{v}{2d}\right) + cm\sqrt{2}$$

or

$$Q = 2\,cm + m^2\left(0.6 - \frac{v}{2d}\right).$$

The intensity of the pressure at the depth m is found by derivation:

$$q = \frac{dQ}{dm} = 2c + 2m\left(0.6 - \frac{v}{2d}\right)$$

hence for $m = o$, $q = 2c$ and for $m = d$, $q = 2c - v + 1.2\,d$.

Fig. 12 e represents the diagram of the resultant loading on the wall.

Taking moments about A:

$$\frac{1}{2}\,\frac{h^2}{3}(v - h) + \frac{h^2 v}{3} + \frac{1}{3}\,dv\left(h + \frac{d}{3}\right)$$

$$= 2\,cd\left(h + \frac{d}{2}\right) - \frac{1}{2}\,d\,(v - 1.2\,d)\left(h + \frac{2}{3}\,d\right), \text{ which gives}$$

$$c = \frac{v}{2} + \frac{h^2(3v - h) - 1.2\,d^2(3h + 2d)}{6d(2h + d)} \qquad \text{(I)}$$

Projection on an horizontal line:

$$R = h\left(v - \frac{h}{2}\right) - d\,(2c - v + 0.6\,d). \qquad \text{(II)}$$

The bending moment at the depth $x \leqq h$ below A is

$$M_x = Rx - (v - h)\,\frac{x^2}{2} - \frac{1}{6}\,x^3 \text{ which is a maximum for}$$

$$x_0 = -(v - h) \pm \sqrt{(v-h)^2 + 2R} \text{ provided } x_0 \leqq h, \qquad \text{(III)}$$

in which case

$$M_{max} = R\,x_0 - (v - h)\,\frac{x_0^2}{2} - \frac{1}{6}\,x_0^3 \qquad \text{(IV)}$$

The bending moment at a distance $x' \leqq$ above C is found

$$M'_x = -0.2\,x'^3 + x'^2\left(c - \frac{v}{2} + 0.6\,d\right) \text{ which is a maximum for}$$

$$x'_o = \frac{1}{0.3}\left(c - \frac{v}{2} + 0.6\,d\right), \text{ provided } x'_o \leqq d. \tag{V}$$

This gives

$$M'_{max} = 0.1\,x'^3_o. \tag{VI}$$

The latter formula is to be used when $x'_o \leqq d$ and this condition will be fulfilled when $\left(\frac{d}{h}\right)^3 \geqq 5\left(\frac{v}{h} - \frac{1}{3}\right)$.

The formulae (I)—(VI) may be represented graphically introducing the ratios $\frac{d}{h}, \frac{c}{h}, \frac{R}{h^2}, \frac{x_o}{h}$ and $\frac{M_{max}}{h^3}$ thus writing the formulae:

$$\frac{c}{h} = \frac{1}{2}\left(\frac{v}{h}\right) + \frac{3\left(\frac{v}{h}\right) - 1 - 1.2\left(\frac{d}{h}\right)^2\left[3 + 2\left(\frac{d}{h}\right)\right]}{6\left(\frac{d}{h}\right)\cdot\left[2 + \left(\frac{d}{h}\right)\right]} \tag{Ia}$$

$\left(\frac{c}{h}\right)$ rectilinear with $\left(\frac{v}{h}\right)$ for $\left(\frac{d}{h}\right)$ = constant.

$$\frac{R}{h^2} = \frac{v}{h} - \frac{1}{2} - \frac{d}{h}\left(2\frac{c}{h} - \frac{v}{h} + 0.6\,\frac{d}{h}\right) \tag{IIa}$$

$\frac{R}{h^2}$ rectilinear with $\frac{v}{h}$ for $\frac{d}{h}$ = constant.

For $\frac{x_o}{h} \leqq 1$ i. e. for $\left(\frac{d}{h}\right)^3 \leqq 5\left(\frac{v}{h} - \frac{1}{3}\right)$:

$$\frac{x_o}{h} = -\left(\frac{v}{h} - 1\right) + \sqrt{\left(\frac{v}{h} - 1\right)^2 + 2\,\frac{R}{h^2}} \tag{IIIa}$$

$$\frac{M_{max}}{h^3} = \frac{R}{h^2}\,\frac{x_o}{h} - \frac{1}{2}\left(\frac{v}{h} - 1\right)\left(\frac{x_o}{h}\right)^2 - \frac{1}{6}\left(\frac{x_o}{h}\right)^3 \tag{IVa}$$

For $\frac{x'_o}{h} \leqq \frac{d}{h}$ i. e. for $\left(\frac{d}{h}\right)^3 \geqq 5\left(\frac{v}{h} - \frac{1}{3}\right)$

$$\frac{x'_o}{h} = \frac{1}{0.3}\left(\frac{c}{h} - \frac{1}{2}\,\frac{v}{h} + 0.6\,\frac{d}{h}\right) \tag{Va}$$

Rectilinear for $\frac{d}{h}$ = constant.

$$\frac{M'_{max}}{h^3} = 0.1\left(\frac{x'_o}{h}\right)^3. \tag{VIa}$$

In Fig. 13 are plotted the curves or lines expressed by the formulae $\mathrm{I}^a$—$\mathrm{VI}^a$.

These curves have been used for designing the sheet piling for the various piers.

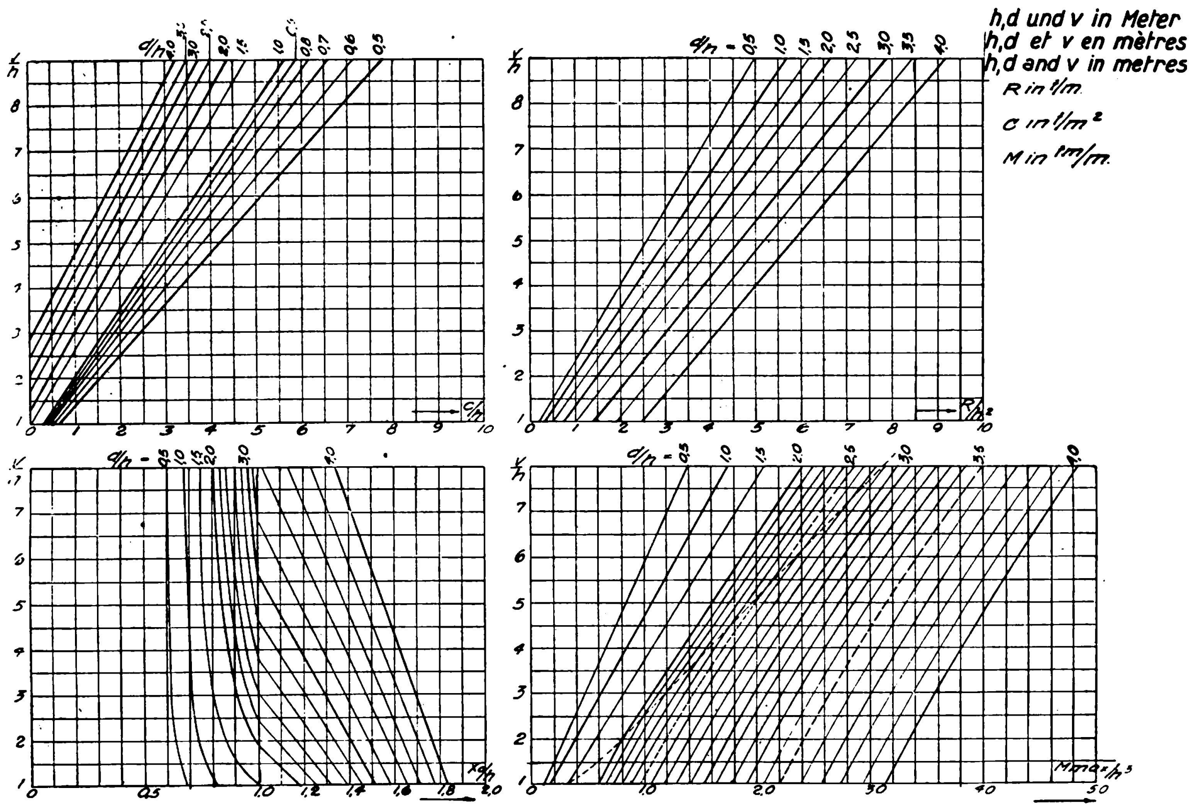


Fig. 13.

Diagram for design of steel sheet piling.

*Measurement of Actual Stresses in Steel Sheet Piling.*

The cross section of the steel sheet piling was in all cases that of Krupp No. III, generally fabricated from ordinary mild steel. Where long piles were needed and the bending moments were high Chromador steel with an ultimate tensile strength of 5800 kg/cm² and a yield point of 3600/cm² was employed.

During the excavation of the pit the bending stresses were ascertained to get an idea of the exactitude of the assumptions made regarding the resistance of the clay and the basis upon which the statical calculations were made.

As some of the sheet piles might be bent during driving, it would have been useless to employ gauge lines laid out on the piles before driving.

As an expedient the curvature of a great number of the piles was measured and the mean value of these curvatures was assumed to express the stresses produced by water pressure etc.

Further the inclinations of the piles were measured by a clinometer.

The apparatus employed is illustrated on Fig. 14, which probably is self-explanatory. When held against a pile the gauge dial will show the rise f of the pile on a length L equal to 1.50 m.

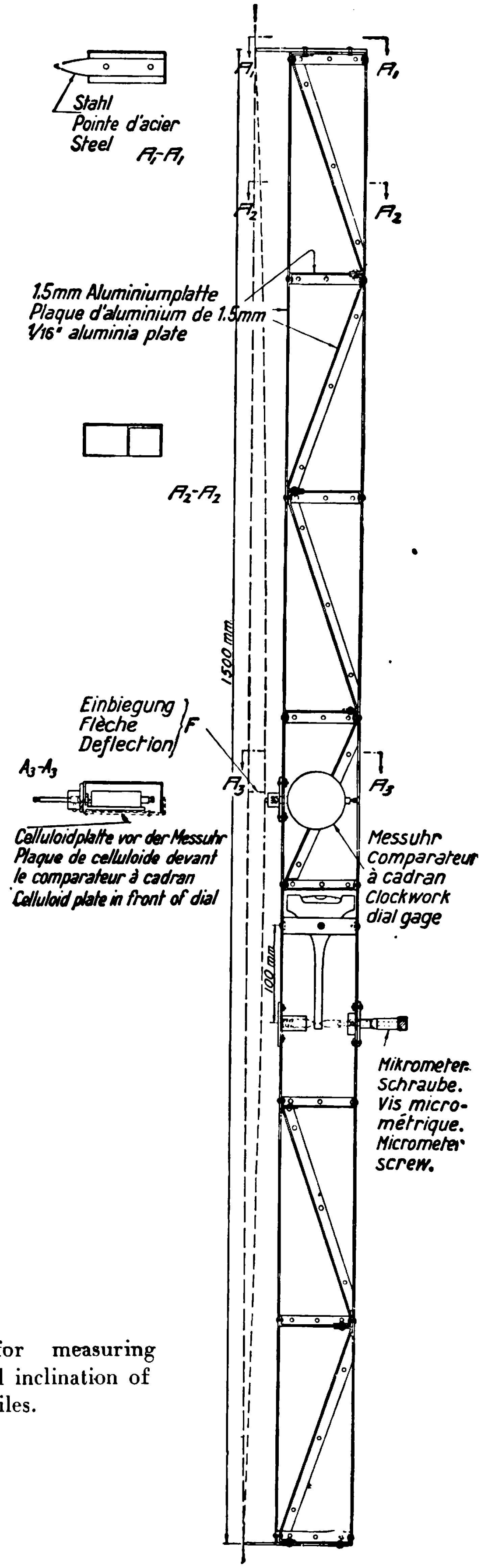


Fig. 14.

Apparatus for measuring curvature and inclination of steel sheet piles.

$$\text{The radius of curvature } \rho = \frac{L^2}{8\,f.}$$

$$\text{The bending moment } M = \frac{EI}{\rho} \text{ where}$$

E = elastic modulus and I = moment of inertia.

Substituting the values for L, E and I and eliminating $\rho$, we find $M = 12.8 \cdot f$, where M is expressed in tm and f in mm.

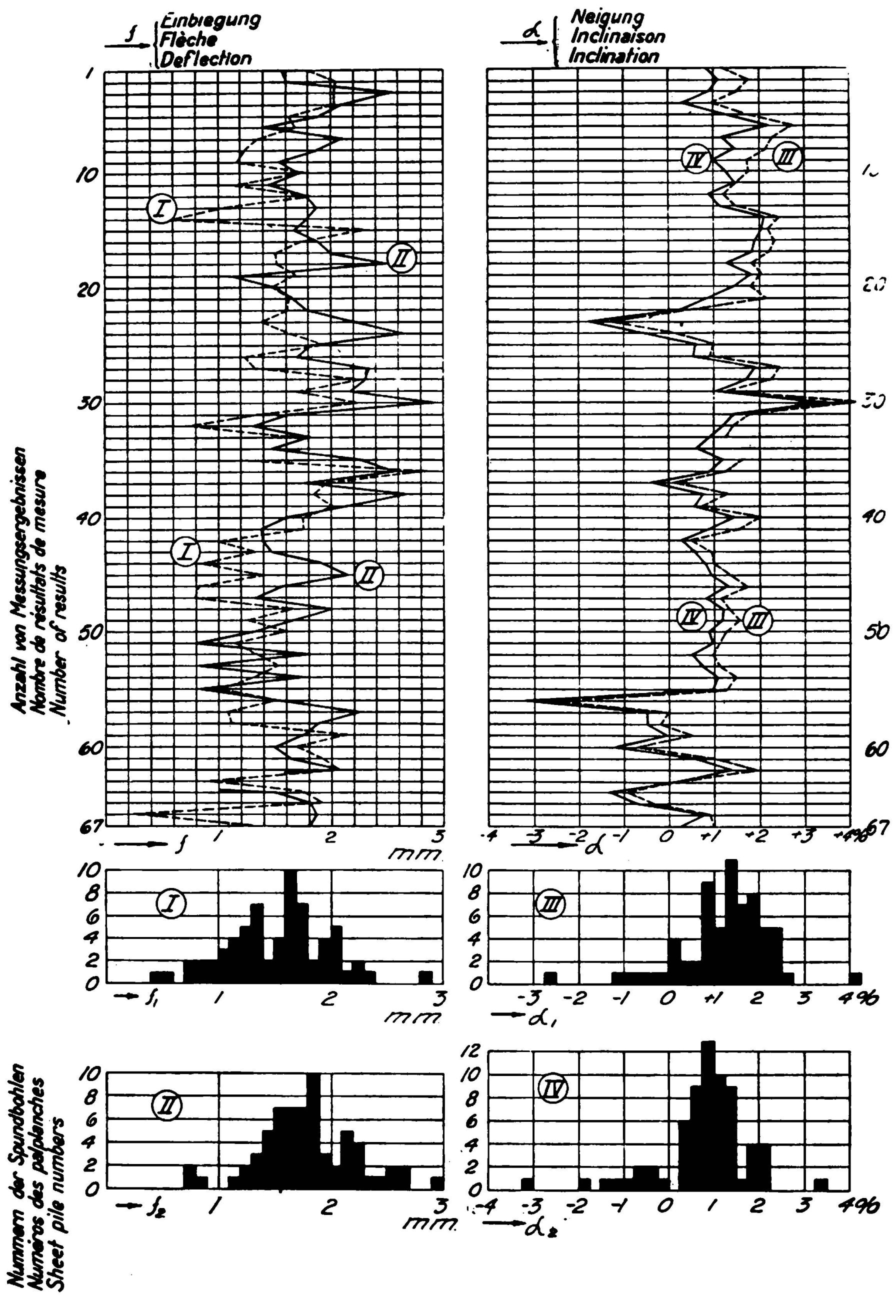


Fig. 15.

Results from check measurements in pier Nr. 39.

I and III are results from measurings betwn. El. — 6.90 m and El. — 8.40 m.

II and IV are results from measurings betwn. El. — 7.65 m and El. — 9.15 m.

The results of a complete observation are shown on Fig. 15. Even in case the individual observations deviate considerably from each other, it is thought that the average will give a fairly good idea of the stresses in the sheet piles.

| PIER Nº PFEILER Nº | PART OF SHEET PILING CHECKED. MESSUNGSSTELLEN | | DATE DATUM. | TOTAL LENGTH OF SHEET PILES TOTALE LÄNGE DER SPUNDBOHLEN | MATERIAL | MEAN CONSISTENCY MITTELKONSISTENZ. | THEORETICAL STRESSES THEORETISCHE SPANNUNGEN $\sigma_{max}$ | $\sigma_2$ AT POINT OF MEASUREMENT $\sigma_2$ IM PUNKTE DER MESSUNGEN | OBSERVED STRESS BEOBACHTETE SPANNUNG $\sigma_2$ |
|---|---|---|---|---|---|---|---|---|---|
| 13 | Total circumference. | Jedes Doppeleisen | 11-6-34 | 8.5 | Mild steel | 5.0 Kg. | 1815 $kg/cm^2$ | 1700 $kg/cm^2$ | 1880 $kg/cm^2$ |
| 39 | " " | " " | 12-6-34 | 7.5 " | " | 10.7 " | 1460 " | 1400 " | 1600 " |
| 9 | " " | " " | 27-6-34 | 7.5 " | " | 7.2 " | 1600 " | 1590 " | 2080 " |
| 14 | " " | " " | 10-8-34 | 8.5 " | " | 6.3 " | 2050 " | 1970 " | 2340 " |
| 8 | " " | " " | 10-9-34 | 11.0 " | Chromador | 5.6 " | 2860 " | 2660 " | 2290 " |
| 5 | Eastern half. | Östliche Hälfte | 26-10-34 | 8.5 " | Mild steel | 8.8 " | 1650 " | 1640 " | 1450 " |
| | Western half. | Westliche " | 27-10-34 | | | 6.1 " | | | 1510 " |
| 40 | Every other double pile | Jedes zweite Doppeleisen | 26-11-34 | 7.0 " | " " | 6.2 " | 910 " | 890 " | 830 " |
| 7 | Western half | Westliche Hälfte | 21-12-34 | 12.0 " | Chromador | 3.5 " | 3610 " | 3150 " | 2110 " |
| 43 | " | " | 21-12-34 | 8.0 " | Mild steel | 7.5 " | 1520 " | 1500 " | 1720 " |
| 26 | Northwestern half | Nordwestliche Hälfte | 27-2-35 | 9.5 " | " " | 8.1 " | 2340 " | 2250 " | 2290 " |
| 42 | " " two thirds | 2/3 d Umfanges N.W. | 23-3-35 | 9.5 " | " " | 5.0 " | 2050 " | 1950 " | 1440 " |
| 17 | Western half | Westliche Hälfte | 27-4-35 | 10.0 " | " " | 6.1 " | 2670 " | 2550 " | 1030 " |
| 41 | " " | " " | 3-5-35 | 12.0 " | Chromador | 2.6 " | 3390 " | 2760 " | 1290 " |
| 16 | " " | " " | 28-5-35 | 12.0 " | " | 2.7 " | 3700 " | 3250 " | 1980 " |
| 44 | " " | " " | 3-6-35 | 9.0 " | Mild steel | 5.1 " | 1810 " | 1660 " | 1400 " |
| 11 | " " | " " | 26-6-35 | 11.5 " | Chromador | 5.9 " | 3260 | 2900 " | 1810 " |
| 46 | " " | " " | 16-7-35 | 8.0 " | Mild steel | 3.3 " | 1290 | 1270 " | 1480 " |
| 10 | Eastern end | Östliches Ende | 1-8-35 | 12.0 " | Chromador | 7.3 " | 3590 | 3150 " | 1920 " |
| | Southern side | Südliche Seite | 3-8-35 | | | | | | 1560 " |
| | Northern side | Nördliche Seite | 3-8-35 | | | | | | 1490 " |
| 45 | Western half | Westliche Hälfte | 8-8-35 | 8.0 " | Mild steel | 6.8 " | 1390 | 1370 " | 1760 " |
| 25 | Eastern half | Östliche Hälfte | 26-8-35 | 10.0 " | " " | 7.5 " | 2540 | 2420 " | 1910 " |
| 47 | Western half | Westliche Hälfte | 30-8-35 | 7.5 " | " " | 5.9 " | 1160 | 1150 " | 920 " |
| 49 | " | " " | 21-9-35 | 9.5 " | " " | 3.8 " | 1770 | 1650 " | 1500 " |
| 37 | Western end | Westliches Ende | 12-10-35 | 12.0 " | " " | 4.7 " | 3620 | 3180 " | 2370 " |
| | Eastern end | Östliches Ende | 14-10-35 | | | 3.0 " | | | 2150 " |
| 48 | Western half | Westliche Hälfte | 17-10-35 | 10.5 | " " | 3.2 " | 2220 | 1940 " | 1490 " |

Fig. 16.

Schedule of check measurements.

All the results from the 24 piers where measurements have been taken are compiled in the table below (Fig. 16).

It should be noted that the theoretical $\sigma_{max}$ and the corresponding $\sigma_2$ at the point where measurements were taken are worked out with the correct water pressure but under the same conditions of support as was employed when calculating the dimensions of the sheeting. As a safety factor of abt. 1.5 has been used in fixing the shearing resistance of the clay it is likely that the bottom end of the sheeting to a certain degree will have been fixed during the measurements and that the assumptions for the calculations will not be quite correct.

As a rule the actual stresses should be expected to be less than the theoretical ones. It would certainly be of interest to figure out more accurately the theoretical stresses under conditions of support more similar to the actual ones, in which case a better accordance would without doubt be found. This can be done without difficulty, and the author has the intention later to carry through such calculations.

It is, however, supposed that the results found will nevertheless be of interest as similar measurements have probably heretofore very seldom been carried out.

In some cases the stresses found by the check measurements were considerably lower than expected. This, of course, may be explained thereby that the bottom conditions were really better than supposed on the basis of the laboratory tests, but it may be mentioned that some of the piers where this has taken place had unusually long sheet piles, and the excavation at the same time was carried out with extraordinary speed, for which reason the plastic deformation of the clay had hardly had time to reach their maximum value. In several cases the author received the impression that the stresses ran higher when the work was, for some reason or other, extended over a longer time.

## Summary.

The standardized method of construction for the foundations of the Storstrømsbridge in Denmark is described.

New hydraulic clay sampler for the extraction of undisturbed samples of the subsoil described.

Method of testing consistency of clay by new cone-apparatus described. Consistency in kilos K, weight of cone G in kilos, penetration of cone y in mm depend on equation

$$K = G \cdot \left(\frac{10}{y}\right)^n.$$

For clay of Storstrømmen n = 1.75.

The cone consistency of fat clays is thought to express the cohesion of the soil without regard to the internal friction.

For the clay in question practical experience makes probable the following relation:

$c = 0.1 \cdot K$ (c in kg/cm$^2$, K in kg).

Statical calculation of steel sheet piling is explained. The clay is supposed to be frictionless during the short period where pit was laid dry. Diagrams for finding driving depth and maximum moments of steel sheet piling established.

Measurement of actual stresses in sheet piling carried out during excavation of pit.

Results compared with theoretical stresses.