VIIa11

FOUNDATIONS FOR THE AGGERSUND BRIDGE, DENMARK

L. BJERRUM,
CHR. OSTENFELD,
W. JONSON,
Copenhagen, Denmark

In the year 1942 a new highway bridge across the Limfjord was opened for traffic at Aggersund, in the northern part of the Jutland peninsula (see Fig. 1). The ground conditions in the bed of the fjord are extraordinarily difficult 1), 2) for foundation purposes and a thorough study of the characteristics of the soil was therefore fundamental in the selection of the correct design of the bridge.

In the following the soil conditions at Aggersund, their influence on the choice of the bridge design and the special pile investigation undertaken prior to and during the erection of the bridge, will be briefly discussed.

The geological classification of the soils and their physical characteristics were determined by Mrs. E.L. MERTZ of "Denmark's Geological Survey".

The physical tests were carried out in the laboratory for soil mechanics of the Danish Royal Technical College. The head of the laboratory, Professor A.F. MOGENSEN, also assisted in computing the settlement of the southern earth approach.

GEOLOGICAL DESCRIPTION.

At the location of the bridge the fjord is about 5-600 m wide, the depth of the water being abt. 12 meters (see Fig. 1). The soil conditions in and about the centre line of the bridge have been thoroughly examined by a large number of borings - mostly 4½" dry borings - and acurately plotted.

The upper stratum of the fjord bottom consists of an alluvial fine sand with shells, the thickness of which varies between 12 and 20 meters and is very loosely packed. Not until a depth of 6-10 m below fjord bottom is reached is any appreciable firmness and compactness exhibited.

Below this deposit of fine sand, postglacial, marine silt is encountered, a most characteristic formation from the Litorina-Sea. This layer is very heterogeneous and stratified with a varying content of clay, silt, seaweed, shell debris and fine sand (see description of strata Fig. 1.). Due to the silt and clay the formation is very loose, contains 45-60% of water, and samples exhibit on compression great bulk reduction. As Fig. 1 shows this this layer has its greatest thickness abt. 20 m below the south abutment.

Solid ground, a cretaceous clay, probably of glacial origin, is not encountered until a depth of 28-38 m below water level is reached. Uppermost this layer is very stony and sandy and compared with the overlying strata must be considered stable.

Beneath the glacial deposits lies the bedrock -the chalk.- Notwithstanding that the borings were carried down to considerable depths and that chalk pits arefound on both sides of the Sound, this formation was not reached.

BRIDGE DESIGN.

The poor bearing capacity of the soil was the determining factor in the choice of bridge design and method of construction.

The passage of shipping necessitated a movable span in the centre of the stream and the considerable depth of water not inviting the use of intermediate piers, it was decided to design the bridge with a bascule-span of 30 m in the centre and 2 statically determinate 90 m arch-spans at the ends. The approaches were built as earth embankments rising to 6 m in height above sea level (see Fig. 1). The bridge thus has 2 piers, where the depth of water is 10-12 m and 2 abutments, where the depth is 2-4 m.

On account of the durability and economical maintenance and also on aesthetic grounds the arch-spans were constructed of reinforced concrete. As this however would necessitate large formwork with unyielding foundations, requiring long and expensive piles, the main reinforcing of the arch-spans was erected as stiff skeletons of rolled sections strong enough to carry the formwork, scaffolds etc. (Melan construction). These skeleton structures were built at a shipyard and lightered to the site. During pouring of the concrete, the spans were temporarily supported at the centre by a falsework on piles, preloaded with sand, which was removed during the concreting (see later). The abutments were built in place, founded on shorter timber piles, while the piers were built on shore as caissons, which were launched, sited and sunk into position on to their foundations of long reinforced concrete piles, cut off level with the bottom of the fjord. (A. Engelund's foundation method 3)).

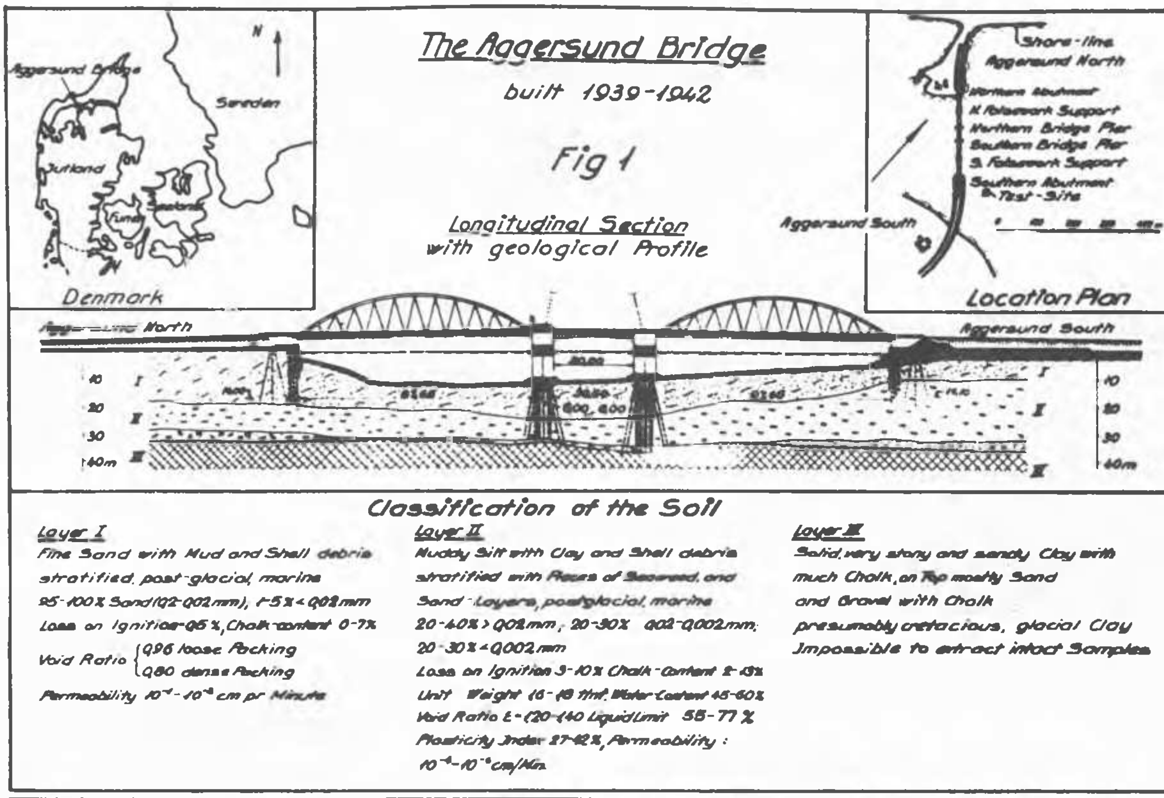
THE BRIDGE PIERS.

As the required precision of a bascule span demands absolute unyielding supports, the piers were founded on 38 x 48 cm reinforced concrete piles carried down to solid ground, their points being about 37 m under water level (see Fig. 2). The piles, each weighing 14 t. were driven from a pontoon piledriver with a 6 t hammer.

The bridge pier caissons were provided at the bottom with an air-lock working chamber in order to permit a solid concrete junction with the pile caps 3). It was desirable to use a minimum number of piles, in order to keep down the dimensions of the piers. The pile loading consequently being comparatively high, a test of one of the vertical piles (see Fig. 2) was made before concreting of the working chamber took place.

As counterpoise for the loading the ceiling of the working chamber was utilized, i.e. the deadload of the pier, which limited the test loading to 120 tons.

The result of this test loading is shown in Fig. 2. The stress diagram for the pile is a straight line up to a maximum load of 120 tons, the settlement being only 2,7 mm. The ultimate resistance of the pile will presumably be about 500 tons, 425 tons friction and 75 tons point resistance. These figures are estimated partly from experience with test piles and partly from several "modelpile experiments" in which by test loading a "miniature-pile" driven at the bottom of a bore tube the friction and the point resistance at different depths were determined 5).


The Aggersund Bridge
built 1939-1942
Fig 1
Longitudinal Section
with geological Profile
Aggersund Bridge
Sweden
Jutland
Funen
Zealand
Denmark
Shore-line
Aggersund North
Aggersund South
Location Plan
Classification of the Soil
Layer I
Fine Sand with Mud and Shell debris
stratified, post-glacial, marine
95-100% Sand (0.2-0.02 mm), 1-5% < 0.02 mm
Loss on Ignition 0.5%, Chalk-content 0-7%
Void Ratio 0.96 loose Packing, 0.80 dense Packing
Permeability 10⁻⁴-10⁻⁵ cm pr Minute
Layer II
Muddy Silt with Clay and Shell debris
stratified with Pieces of Seaweed, and
Sand-Layers, postglacial, marine
20-40% > 0.02 mm, 20-30% 0.02-0.002 mm,
20-30% < 0.002 mm
Loss on Ignition 3-10% Chalk-Content 2-13%
Unit Weight 1.6-1.8 t/m³ Water Content 45-60%
Void Ratio ε = 1.20-1.40 Liquid Limit 55-77%
Plasticity Index 27-42%, Permeability:
10⁻⁴-10⁻⁶ cm/Min.
Layer III
Solid, very stony and sandy Clay with
much Chalk, on Top mostly Sand
and Gravel with Chalk
presumably cretacious, glacial Clay
Impossible to extract intact Samples

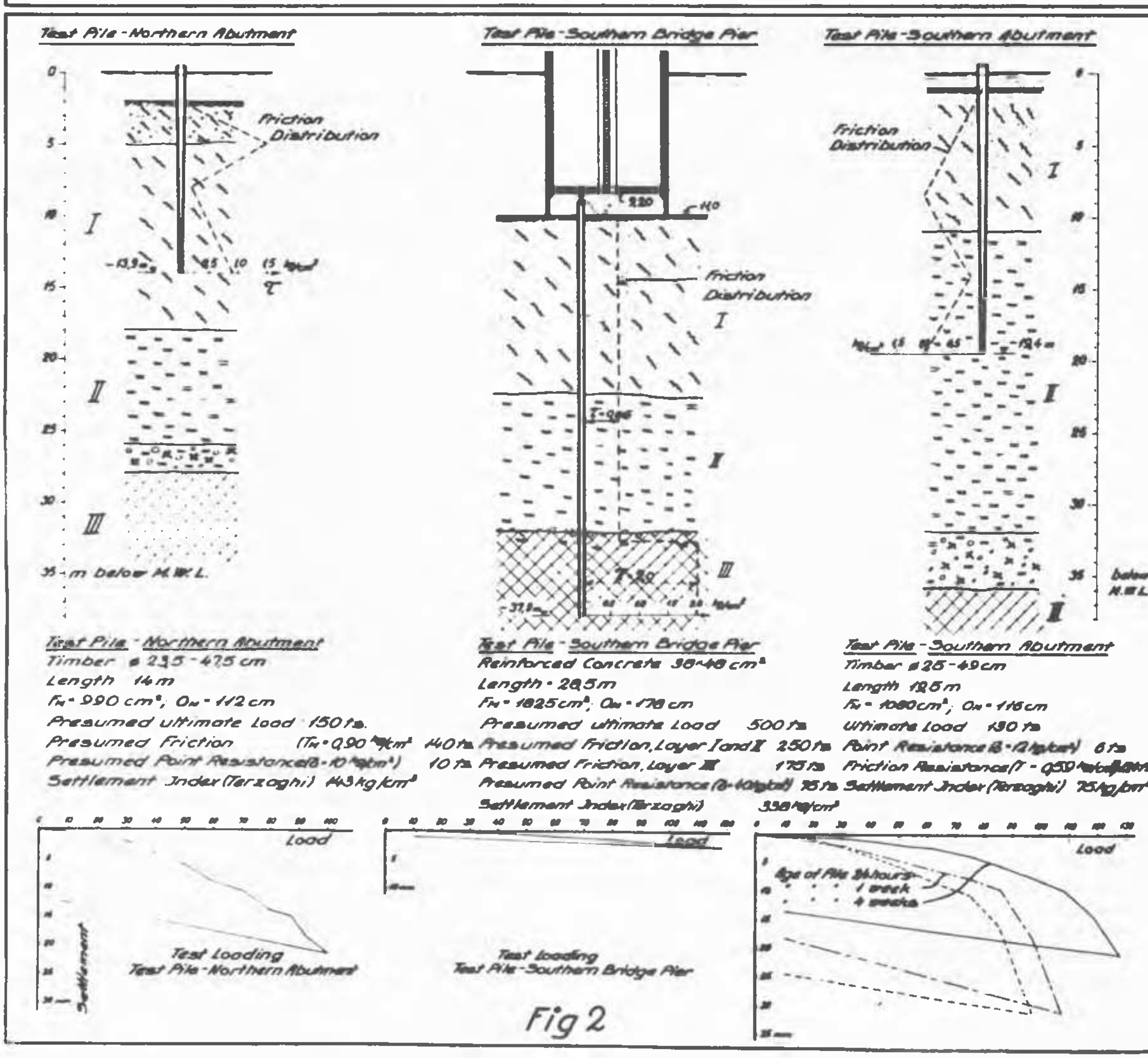

Test Pile - Northern Abutment
Test Pile - Southern Bridge Pier
Test Pile - Southern Abutment
Friction Distribution
m below M.W.L.
Test Pile - Northern Abutment
Timber ø 23.5 - 47.5 cm
Length 14 m
F = 990 cm²; O = 112 cm
Presumed ultimate Load 150 ts.
Presumed Friction (T = 0.90 kg/cm²) 140 ts
Presumed Point Resistance (B = 10 kg/cm²) 10 ts
Settlement Index (Terzaghi) 143 kg/cm³
Test Pile - Southern Bridge Pier
Reinforced Concrete 38×48 cm²
Length = 28.5 m
F = 1825 cm²; O = 178 cm
Presumed ultimate Load 500 ts
Presumed Friction, Layer I and II 250 ts
Presumed Friction, Layer III 175 ts
Presumed Point Resistance (B = 40 kg/cm²) 75 ts
Settlement Index (Terzaghi) 338 kg/cm³
Test Pile - Southern Abutment
Timber ø 25 - 49 cm
Length 19.5 m
F = 1080 cm²; O = 116 cm
Ultimate Load 130 ts
Point Resistance (B = 12 kg/cm²) 6 ts
Settlement Index (Terzaghi) 75 kg/cm³
Load
Settlement
Age of Pile 24 hours
1 week
4 weeks
Test Loading
Test Pile - Northern Abutment
Test Loading
Test Pile - Southern Bridge Pier
Fig 2

FIG. 1, 2

Fig. 2 shows the distribution of the frictional resistance in these experiments.

THE ABUTMENTS. (Fig. 3 and 4).

Completely unyielding foundations for the abutments would have called for reinforced concrete piles abt. 35 m long driven into the solid glacial clay (layer III). A foundation of this kind being, however, an economic impracticability the abutments were founded on timber piles abt. 16 m long, i.e. with about 14-20 m fine silt below the pile points. The piles were thus relied upon, by side friction to distribute the reaction from the arch spans over so large an area that settlement of the abutments caused by compression of the soil beneath the pile points does not exceed that allowed for in the super. structure. By building the abutments higher than design level it was possible to secure the required final longitudinal profile. This subject will not be further dealt with here, as it will be fully discussed in a special paper, now in preparation.

The foundation for the abutments was designed with 4 frontal rows of inclined piles to take the reaction (abt. 620 tons), while a pile trestle system in the rear, anchors the abutment to the earth approach (see Fig. 1 and 4).

In both of the abutments one of the vertical piles was test loaded. Fig. 2 shows the test piles and their dimensions together with the results of the tests. As in the case of the test pile in the southern bridge pier, the assumed distribution of the frictional forces over the sides of the piles is given. In loading the test pile in the northern abutment failure was not achieved. With a loading of 100 tons the settlement of the pile was 22 mm. The ultimate resistance of the pile was estimated at 150 tons.

The test pile in the southern abutment was loaded 4 weeks after driving and showed an ultimate resistance of 130 tons. Realizing that the bearing power of the pile increases with time it was decided to investigate this question further. By re-driving the pile was "aroused" and the following day again testloaded. The bearing power had then decreased to 97 tons.

One week later the pile was again loaded and exhibited a bearing power of 108 tons. The result can be thus tabulated:

the age of the pile	0 week bearing	power 97 t, τ= 0,45 kg/cm2
- - - - - -	1 - -	power 108 t, τ= 0,50 kg/cm2
- - - - - -	4 weeks -	power 130 t, τ= 0,60 kg/cm2

During construction of the southern abutment the test pile was carried up through the abutment but free of contact with it (Fig. 7 shows the arrangement of the abutment with the test pile). It is thus possible to compare the settlement of the abutment with the settlement of a single pile and at any time to testload the pile and determine its bearing capacity at progressive age periods.

In order to estimate the settlement of the abutments an attempt was made to calculate, from the test pile results, the stress distribution in the earth beneath the pile points, caused by the pile friction. On the basis of the friction distribution shown in Fig. 2, the stress distribution on the earth layers around a single pile was determined (the "pressurebulb" 6)). Fig.3 showns the result for one of the piles in the southern abutment. The stress distribution in the plane of the pile points (see Fig. 3) could then be summated. As shown the piles have at a depth of 16,10 m distributed the reaction over an elliptical area of abt. I450 m^2, the stress distribution being "bell-shaped" with a maximum of about 0,14 kg/cm^2 beneath the centre of the abutment.

The Approaches.

The road approaches to the bridge were built as 5-6 m earth embankments with a crown width of 11 m and a slope of 1, 5 : 1. It was to be expected that these embankments would suffer heavy settlement. To estimate the factor of safety against failure and the compression of the soft earth beneath, the stress conditions under the embankment were calculated by Boussinesques formula. Fig. 4. shows the longitudinal and cross sections of the southern approach and the stresses arisen.

It will be seen that the greatest shearing stress is to be found in the upper sand layer (layer I) at a depth of 8-9 m. On the basis a settlement calculation was made, the compressibility of the soil being determined upon intact samples.

Fig. 5 shows time-settlement curves for 3 different cross sections of the southern approach embankment. The curves were plotted from the level readings of a number of settlement gauge rods, built into the embankment, having base plates resting directly on the fjord bottom. The diagram also gives the loads of the 3 cross sections. As shown the embankment was carried up 2 meters above design level, to hasten settlement and the compression of the soft layers of fine silt in order that the embankment should as far as possible come to rest before the building of the abutments. The diagram also indicates sufficiently close conformity between the calculated and the measured settlement.

The calculations assume the total weight of the embankment applied at once, while in reality it proceeded very slowly. It emerges from the computations that less than 5 % of the settlement is due to the sand layer (layer I) the remaining 95 % being due to layer II, the fine silt.

Fig. 6 shows the settlement curves for 2 cross sections of the northern abutment.

Falsework supports.

As above mentioned the arch-spans were temporarily supported at the centre during the concreting by timber falsework on piles.(see fig. 8). There were 16 inclined piles abt. 19 m long, driven in groups of 4 at each of the four supporting-points. The depth of water here was about 8-9 m. The falmework at the top was formed as a box, filled with sand, giving a preloading of 330 tons representing the load due to the newly-poured concrete. During the pouring the sand load was successively removed ensuring a constant pile load during the process. The settlement of the supporting piles in relation to time is shown on Fig. 8. While concreting the arches, no measurable settlement was observable, but stress-measurements made at the same time indicated that stresses in the chords of the bridge were higher and in the web members were lower than calculated. These facts show that the supporting piles had not acted as a solid support, but had yielded, the load becoming thus somewhat reduced.

Test-site.

Close to the centre lin of the bridge (see Fig. 1) a test site was established with 9 driven piles. The purpose being to study the actions of the friction piles in the soft bottom of the fjord, under untrammelled conditions - i.e. laboratory tests on a larger scale (7). Fig. 9 shows in tabular form the primary results of these tests. The ground conditions at the test site were approximately similar to

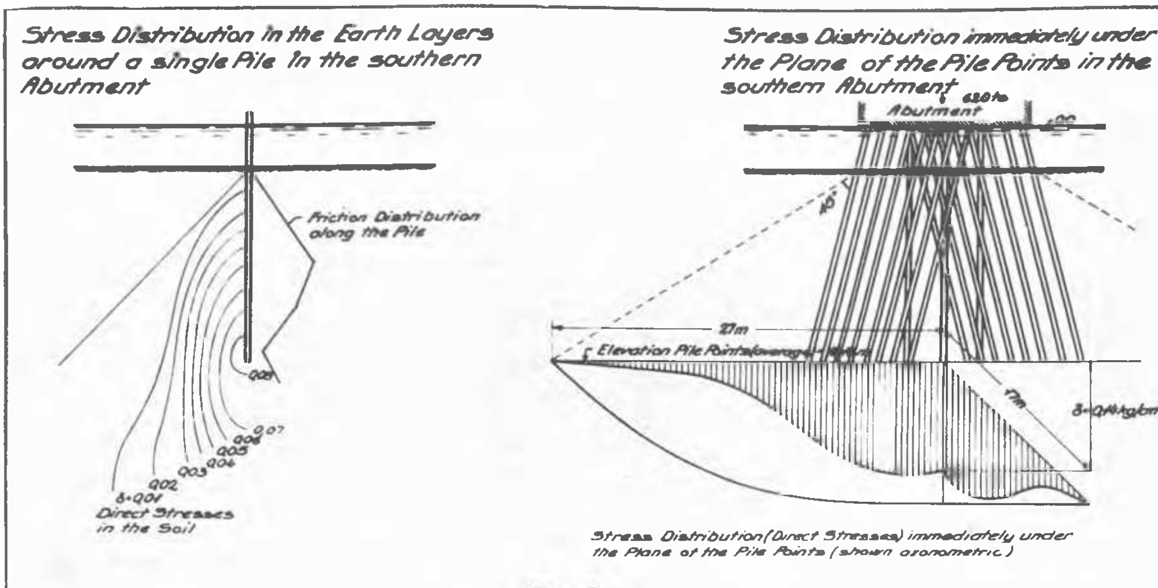


Fig 3

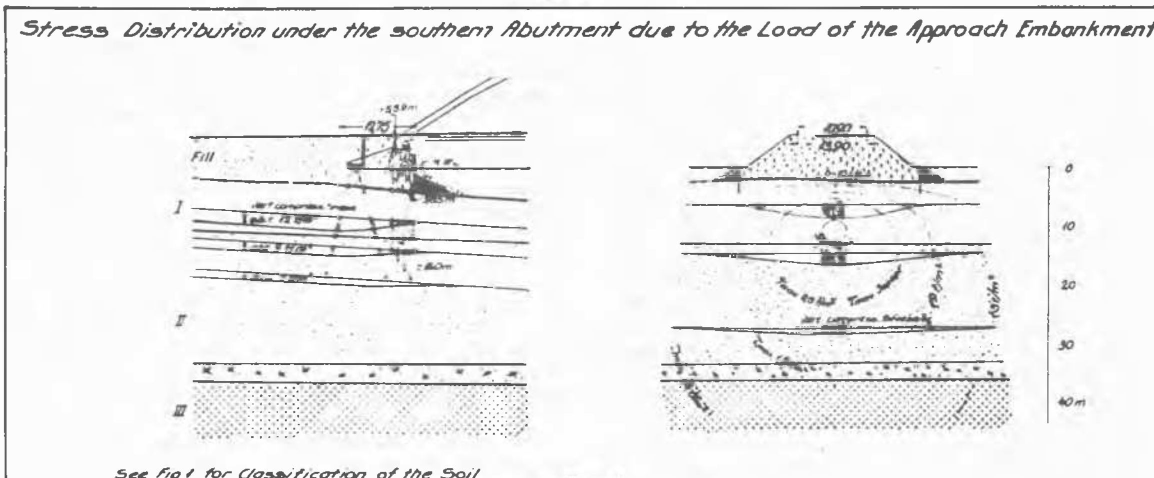


Fig 4

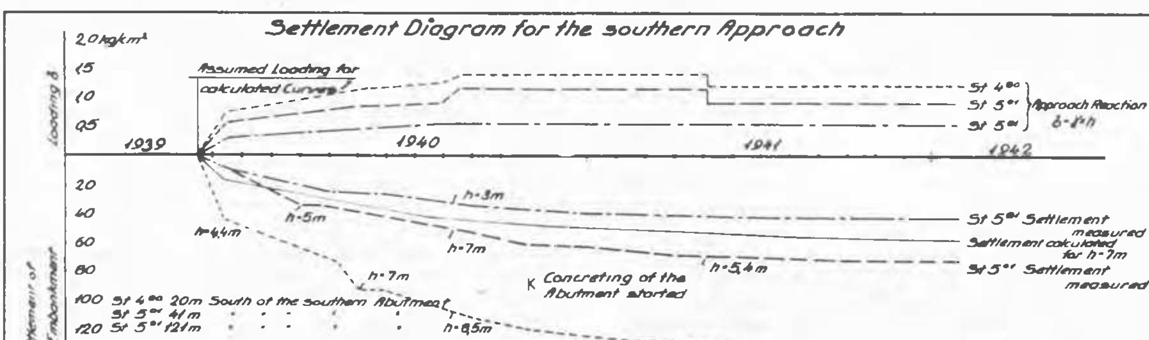


Fig 5

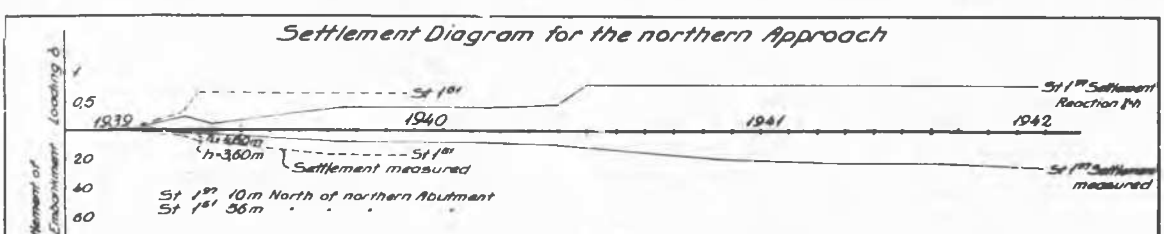


Fig 6

FIG. 3, 4, 5, 6

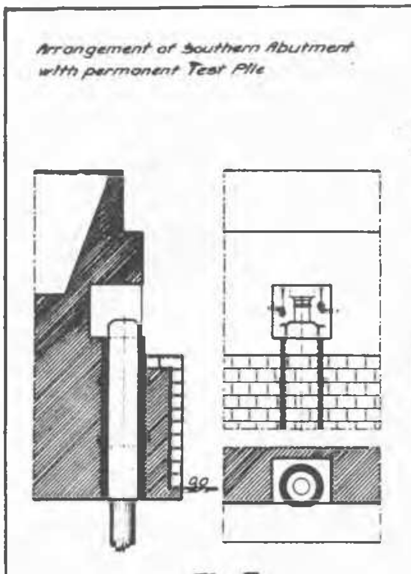


Fig 7

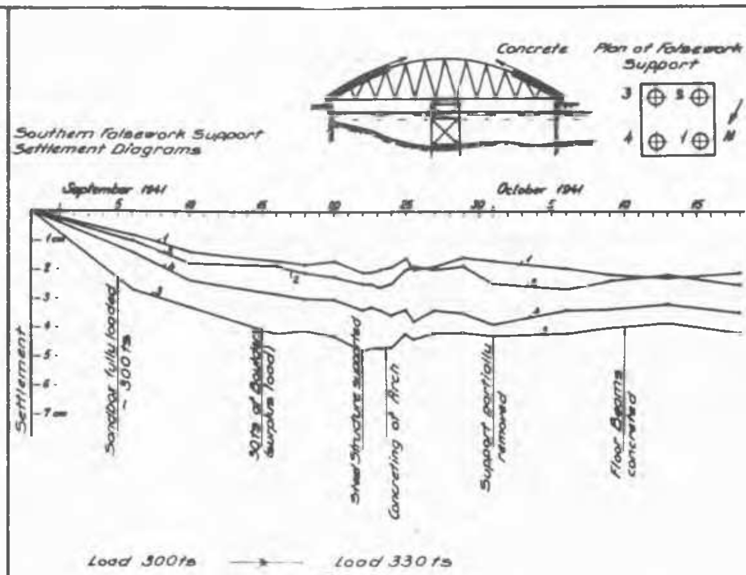
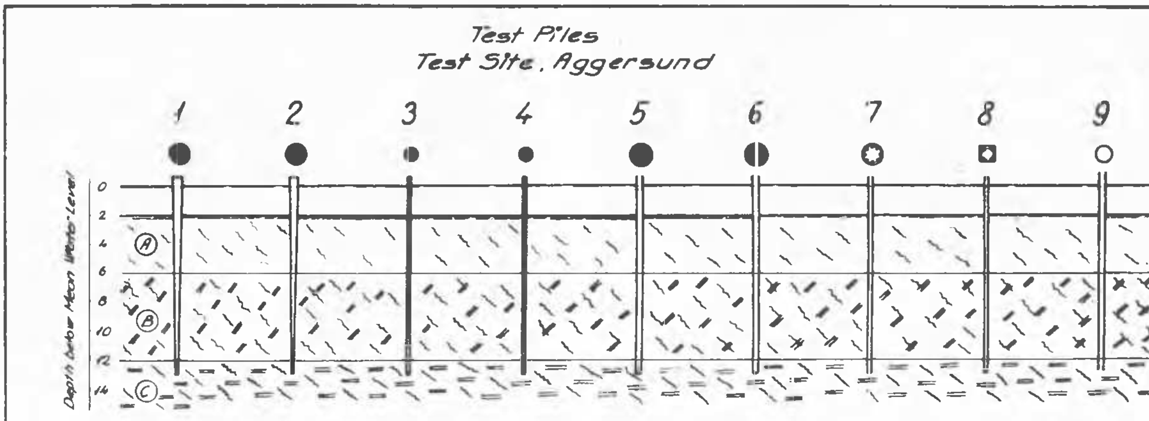


Fig 8



Material	Timber	Timber	Timber planed	Timber planed	Timber planed	Timber planed	Reinforced Concrete	Reinforced Concrete	Steelpipe with Piston
Dimension	ø 22-37cm	ø 22-37cm	ø 22 cm	ø 22 cm	ø 37cm	ø 37cm	ø 32 cm	26·26 cm	Steelpipe ø 26,5 Piston ø 24
Area of Point F / Area of Surface O	F = 380 cm² O = 9,6 m²	380 cm² 9,6 m²	380 cm² 7,5 m²	380 cm² 7,5 m²	1080 cm² 12,6 m²	1080 cm² 12,6 m²	805 cm² 11,2 m²	675 cm² 11,6 m²	455 cm² 9,0 m²
Ultimate Strength Compression	31,5 ts	31,2 ts	20,8 ts	20,4 ts	34,8 ts	34,8 ts	31,2 ts	31,5 ts	Steelpipe empty [illegible] Piston [illegible]
Ultimate Strength Tension	30,4	27,2	13,3	16,5	28,0 -	30,4 -	30,4 -	27,2	Steelpipe 12,6 ts
Ultimate Strength Compression without [illegible]	31,2	27,2	17,2	16,1	27,6 -	31,2 -	27,2 -	28,0	
Friction Resistance	30,8 ts	27,2 ts	15,3 ts	16,3 ts	27,8 ts	30,8 ts	28,8 ts	27,6 ts	14,9 ts
Point Resistance	0,7	4,0	5,5	4,1	7,0	4,0 -	2,4	3,9	5,4
τ t/cm²	0,32	0,28	0,21	0,22	0,22	0,25	0,26	0,24	0,17
δ Point	1,8	10,5	14,5	10,8	6,5	3,7	3,0	5,8	11,9
Average τ	0,30		0,22		0,23		0,25		0,17
Average δ	9,4				5,1		3,0	5,8	11,9
Bearing Capacity according to formulas: Weisbach	22,8 ts	26,5 ts	17,8 ts	26,4 ts	27,2 ts	36,2 ts	14,6 ts	13,2 ts	
Bearing Capacity according to formulas: Brix	8,9 -	10,3 -	4,9 -	7,2 -	13,1 -	17,5 -	10,4 -	8,9 -	
Bearing Capacity according to formulas: Engineering News	17,5 -	18,8 -	14,0 -	17,2 -	20,1 -	22,3	19,6 -	17,9 -	
Actual Bearing Power	31,5 -	31,2 -	20,8 -	20,4 -	34,8 -	34,8 -	31,2 -	31,5 -	

Ground conditions of the test site.Classification of the soil

Layer A 200 - 6,10 m below 0,0
Very fine sand with mud and shells, postglacial,marine 100 % sand (0,2 - 0,06 mm) 0 % silt 0 % clay loss on ignition 8 %,chalk-content 8% , unit weight 183, void ratio ε = 0,89 (loose packing) - 1,08 (dense packing).

Layer B 6,10 - 11,80 m below 0,0
Fine sand with a little clay, seawed shells, postglacial,marine 90 % sand (0,2 - 0,06 mm) 10 % silt and clay loss on ignition 0,4 %,chalk - content 2 %,unit weight 1,82,water - content 27 - 35 %,void ratio ε = 1,20.

Layer C 11,80 - (21,0) m below 0,0
Greasy, muddy clay, with a little sand, shells, postglacial, marine 50 % 0,02 cm abt. 25 % silt abt. 25 % clay loss on ignition 2 - 8 %,chalk content 4 - 11 %,unit weight 1,67,water-content 41 - 63 %, void ratio ε = 1,55.

FIG. 7, 8, 9

those on the centre line of the bridge. The layers were, however, somewhat more loosely packed. The water depth was 2 m and all 9 piles were driven to 12,90 m below water level.

The 9 piles consisted of the following:
2 conical wooden piles Ø 22-37 cm (planed)
2 cylindrical - - Ø 22 cm "
2 - - - Ø 37 cm "
1 cylindrical reinforced concrete pile Ø 32 cm
1 rectangular - - - 26x26 cm
1 cylindrical steel pipe Ø 26,5 cm.,

The pipe was emptied after driving and the pile and resistance at the various depths measured by means of a piston Ø 24 cm. None of the piles were pointed at the lower end. A few months after driving, the piles were pressure loaded until failure occurred. Some weeks later the piles were put under tension, and immediately thereafter again pressure loaded. While the first experiment determined the friction-resistance plus point-resistance, only friction resistance is effective in the tension test and the closely following pressure loading. Prior tests had established that the bearing power of the pile was uninfluenced by age. It was thus possible to calculate the friction resistance and the end resistance and the average τ and σ values. Fig. 9 shows these values, and it will be seen that the friction stress τ is independant of the cross sectional form and the size of the pile. On the other hand τ varies with the pile material, being least for the steel pipe and somewhat greater for the reinforced concrete pile than for the timber pile. As could be expected τ is greater for the conical piles than for cylindrical piles. The computed σ-values seem to show that the point-resistance diminishes with the dimensions of the point.

From the loading tests on the 9 piles it is possible to judge the reliability of the driving formulae for the soils encountered (friction piles). Eytelwein's formula gives the most a applicable results, while the Engineering-News and Brix formulae give values 40-60% too low.

These pile experiments made at the test site are more fully treated in Chr. Ostenfeld's "Tests with Piles" 5).

REFERENCES.

1) Mertz, E.L. Geologiske Profiler gennem danske Sunde og Fjorde (Geological Profiles through Danish Sounds and Fjords). Danmarks Geologiske Undersøgelser II Række Nr.60
2) Kierulff, O. Limfjordsbroen (The Limfjord Bridge). Bygningsstatiske Meddelelser 1942 Nr. 3.
3) Engelund, A. A Special Foundation Method for Bridge Piers, Adopted in Danish Fjords. Proc. Int. Conf. Soil Mechanics 1936, Vol. I.
4) Bretting, A.E. Soil Studies for the Storstrøm Bridge, Denmark. Proc. Int. Conf. Soil Mechanics 1936, Vol. I.
5) Ostenfeld, Chr. Forsøg med Pæle (Resumé en Francais) (Tests with Piles). IngeniørvidenskabeligeSkrifter 1942 Nr. 3.
6) Schenck, W. Zu Frage der Tragfähigkeit von Rammpfählen. Die Bautechnik, H. 53/54 1938.
7) Müller, R. Modellversuche über das Zusammenwirken von Mantelreibung, Spitsenwiderstand und Tragfähigheit voh Pfählen. DEGEBO, Mitteil. No 7 1939

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VIIa12 RESULTS OF A SPECIAL LOADING-TEST ON A REINFORCED CONCRETE PILE, A SO-CALLED PILE SOUNDING; INTERPRETATION OF THE RESULTS OF DEEP-SOUNDINGS, PERMISSIBLE PILE LOADS AND EXTENDED SETTLEMENT OBSERVATIONS

G. PLANTEMA

Engineer of City Engineers Dept., Sub-Dept. of Soil Mechanics, Rotterdam

SUMMARY

From investigations of deep-soundings, performed in the field, the required length and toe-area of foundation-piles can be concluded. A special pile-sounding test has been made, whereby the toe resistance at various depths is determined, eliminating at the same time the skin friction, and compared with the results of deep-soundings. The test and its results are described in the following report, analysing the sounding-resistance with regard to ultimate bearing capacity (loss of equilibrium) and ratio between load and penetration of a pile toe, which is resting in sand layers. Extended settlement observations have been made and increased settlement of the sand under the pile-toe registered.

I INTRODUCTION:

For pile foundations it is important to know the depth to which piles must be driven and what toe area is required for a certain load. For this purpose the bearing capacity of the subsoil, especially the sand-layers carrying the foundations, must be known. During the last decade results from deep-soundings have been used, whereby a cone of a sectional area of 10 cm^2 is pressed into the earth (for a description of the apparatus see 1) and 2). The resistance of the cone at various depths is ascertained; the relation between depth and resistance, plotted in a diagram will be an indication for the bearing capacity of the subsoil.

From this diagram the most favourable foundation depth and toe-area can be determined, taking into account the negative skin friction and a factor of safety. It is hereby as-