



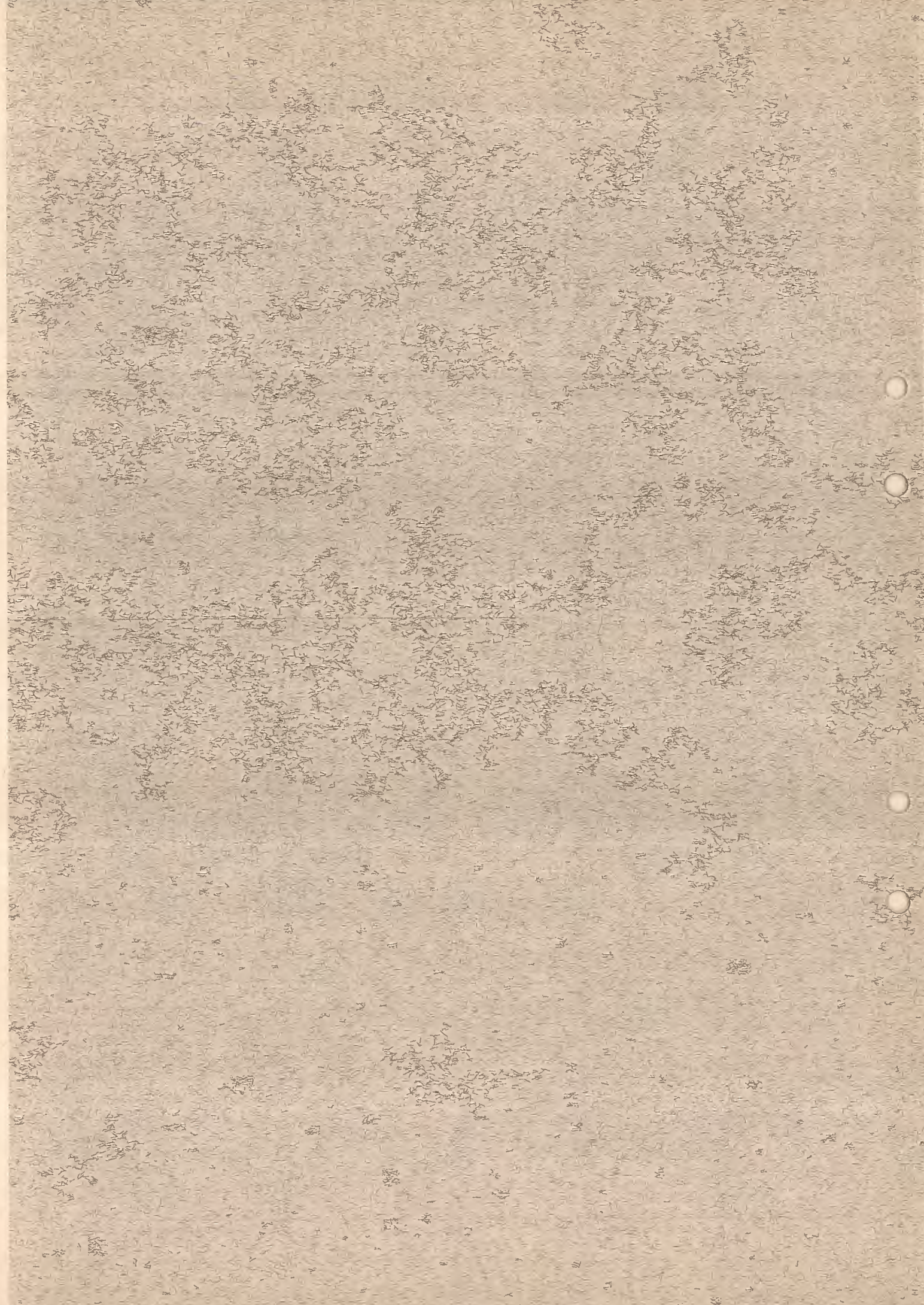
STATENS GEOTEKNISKA INSTITUT

SPREAD FOOTINGS IN SANDS. CALCULATION OF
BEARING CAPACITY AND SETTLEMENTS

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SPREAD FOOTINGS IN SANDS. CALCULATION OF BEARING CAPACITY AND SETTLEMENTS

SYNOPSIS

The Swedish National Road Administration and the Swedish Geotechnical Institute have investigated how to calculate the settlements for bridge footings in cohesionless soils. A series of load tests have been performed at two test sites using 0.6-2.5 m concrete slabs. The soil consisted of medium dense to dense silty sand at one site and loose to very loose sand at the other. Field investigations consisted of CPT-tests, SPT-tests, weight sounding tests, pressuremeter tests and sampling.

The settlement calculations were performed in accordance with De Beer (CPT), Schmertmann (CPT), Schultze-Sherif (SPT), Parry (SPT), Ménard (pressuremeter) and using elastic theory and compression moduli from oedometer tests. The results indicate that settlements of foundations in cohesionless soils can be calculated from static or dynamic penetration tests. The correlation is best for large footings. Measurements from the pressuremeter method result in too small theoretical settlements in dense soil compared to those obtained in practice after a long time.

The methods used for calculating the bearing capacities of the slabs show a large scatter. The Menard method sometimes overestimates and sometimes underestimates the ultimate bearing capacity. The other methods based on SPT- and CPT-test results underestimate the bearing capacity by a factor of 2-5.

INTRODUCTION

The Swedish Geotechnical Institute (SGI) and the Swedish National Road Administration (SNRA) have during recent years carried out investigations into the calculation of bearing capacities and settlements of spread footings in cohesionless soils. The aim of this investigation is to evaluate the different investigation and calculation methods for footings in sands. The results of two fullscale bridge tests with settlement measurements have been reported earlier (Bergdahl and Ottosson, 1982). As there were some difficulties in evaluating the real foundation pressures and measuring the real settlements on these bridges it was decided to perform two series of load tests on rather large concrete slabs 0.6-2.5 m. This paper summarizes the results of these latter investigations.

TEST SITES

The first test site is located at the SGI Kolbyttemon test field, about 10 km south of Linköping in a glaciofluvial sand deposit. The second test site is also located in a glaciofluvial sand deposit, at Fittja, close to the Alby lake southwest of Stockholm.

FIELD AND LABORATORY INVESTIGATIONS

Extensive site investigations and laboratory investigations have been carried out on both test sites.

- Cone penetration tests (CPT) in accordance with the recommended European standard.
- Weight sounding tests (WST) in accordance with the recommended European standard.
- Standard penetration tests (SPT) in accordance with the recommended European standard with a free-falling hammer and a height of fall of 0.76 m.
- Dynamic probing in accordance with the Swedish geotechnical HfA standard.
- Pressuremeter tests with a standard Ménard $\phi 60$ mm probe in bore holes stabilized with bentonite slurry.

At Kolbytte the soil consists of sand, silty sand, silt, sandy silt and gravelly sand. The groundwater table was about 8.5 m below ground level. The CPT-results indicate a medium dense to dense soil. The triaxial tests showed angles of internal friction of between 34.5° and 36.0° . The oedometer tests gave secant moduli for the relevant stresses of between 5 and 10 MPa which seems to be too low, probably due to soil disturbance during sampling.

At Fittja the soil consists of silt, sand, silty sand, gravelly sand and sandy silt. The groundwater table was 1.4 m below ground level. The CPT results indicate a very loose to loose soil. Unfortunately there are no results from oedometer or triaxial tests on material from Fittja because of the lack of good quality samples.

PLATE LOAD TESTS ON SLABS

The load tests were performed on four different concrete slabs of average width 0.6, 1.2, 1.7 and 2.4 m at both test sites. The slabs were made slightly rectangular to guide the failure zone to aside from the centre line (Fig. 3).

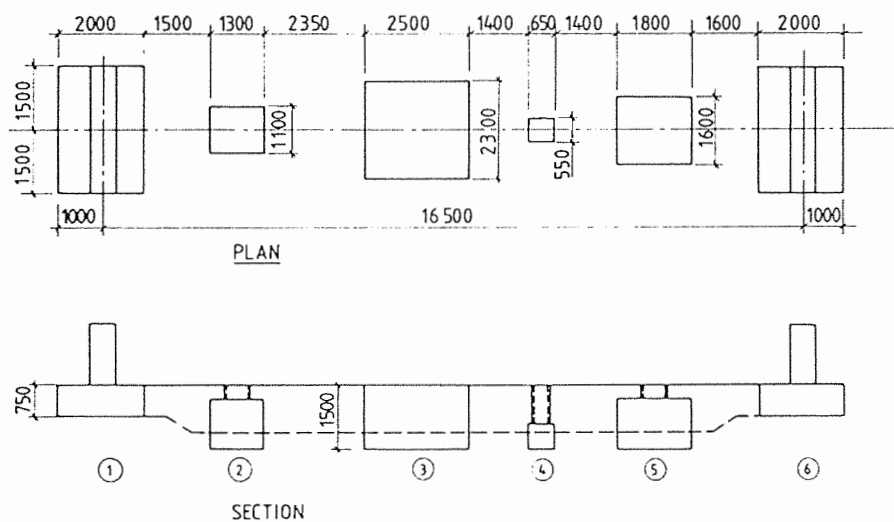


Fig. 3 Test slabs and abutments for the Kolbytte counterweight (mm).

- $\phi 50$ mm piston sampling.
- Laboratory oedometer and triaxial tests on samples from the Kolbytteimon test site.

The HfA type dynamic probing is somewhat different from the recommended European DPB standard. Thus the height of free fall is 0.50 m and the cross-sectional area of the point is 16 cm². Skin friction along the rods is separated from point resistance either by means of a slip coupling or by measuring the torque required to turn the rods (Bergdahl and Möller, 1981).

SOIL PROFILES

The results of the site investigations are summarized in Figs. 1-2.

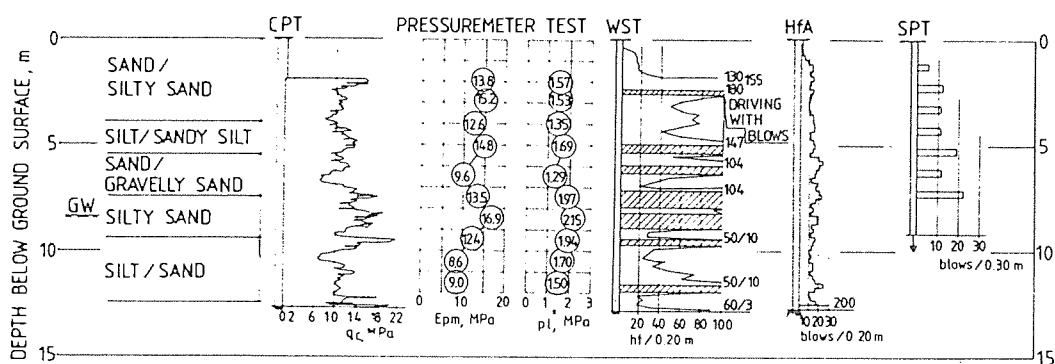


Fig. 1 Results of the Kolbytteimon Field investigations.

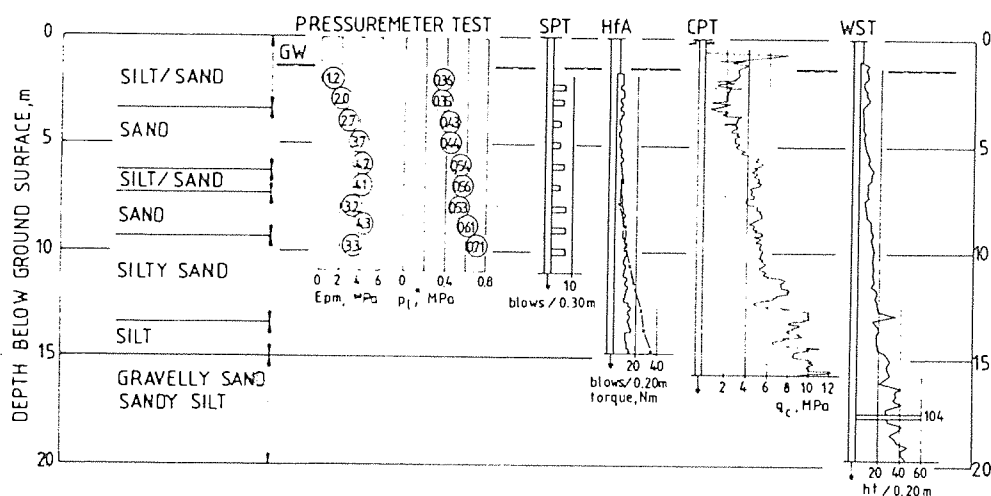


Fig. 2 Results of the Fittja field investigations.

The reaction for load tests was provided by a 3300 kN counter-weight of concrete piles (Fig. 4). This load was supported by two abutments with foundation slabs 2.0 x 3.0 m at Kolbyttemon and 2.4 x 3.6 or 2.8 x 4.2 m at Fittja.

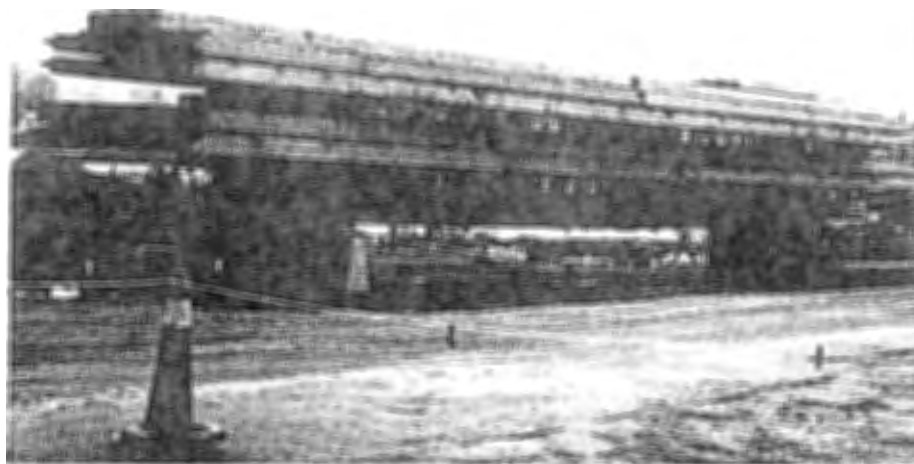


Fig. 4 Plate load test arrangements at Fittja.

Before each load test the soil was replaced to give an embedment depth of $0.65 \times B_m$ (B_m =average width of the slab) to a distance of at least $2 \times B_m$ from the edge of the slab. The load tests were performed using a hydraulic jack with a maximum capacity of 3000 kN and an oil pump with a constant pressure cell. The load was at first increased in steps to a load corresponding to the allowable bearing pressure according to the SIRA Code. (0.11, 0.22, 0.32, 0.46 MPa at Kolbyttemon and 0.04, 0.09, 0.13, 0.18 MPa at Fittja for the slabs of average width 0.6, 1.2, 1.7 and 2.4 m). After each load increment the load was kept constant for 16 minutes in all tests except one. When testing the smallest slab at Fittja the load was kept constant during 60 minutes. At certain load levels cyclic loading and unloading was performed between the maximum load and 50% or 70% of that value. The load was kept constant for 8 minutes at both load levels during these tests.

During the tests settlement readings were taken at the top of the slab and at two or four different levels below the foundation level ($0.75 \times B_m$ and $1.5 \times B_m$ at Kolbytte-mon and at $0.4 \times B_m$, $0.6 \times B_m$, $1 \times B_m$ and $2 \times B_m$ at Fittja). The settlements below the slabs were registered as those of flat screw augers driven to the desired depths. In addition, abutment settlements were measured at both sites during the whole test period. The measuring system consisted of an electronic load cell and a number of potentiometric transducers. Data logging was performed using a small HP 85 computer and a μ Mac 4000 type Analog Devices scanner.

BEARING CAPACITY CALCULATIONS

The allowable and ultimate bearing pressures (q_a , q_u resp.) for the different slabs have been calculated using the following methods:

- The Swedish Building Code, SBN -80, where q_a is dependent on the type of soil and the density according to the WST (Dahlberg, 1974). For Kolbytte-mon the bearing capacity factor for dense fine sand was used and for Fittja that for loose fine sand.
- The SNRA Code, TB 103, where the bearing capacity factor, $k = 8.5$, was selected for dense sand - fine sand in the Kolbytte-mon case. For Fittja k was taken as 4 for medium dense fine sand. The definitions of loose, medium dense and dense are not the same in these two Swedish codes and they do not correspond to international practice.
- For Kolbytte-mon the Danish Foundation Code, DS 415, was also used, with an angle of internal friction of $\phi = 35^\circ$. The partial safety factors chosen were 1.2 for $\tan\phi$ and 1.1 for the applied load.
- The method proposed by Meyerhof (1956), based on CPT.
- The method proposed by Meyerhof (1956) based on SPT. The dynamic probe test results (HfA) were also used in these cases. As pointed out by Bergdahl and Ottosson (1984), the net blowcount in blows/0.2 m of penetration (N'_{20}) for the

HfA test in sands is the same as the N_{30} value for the SPT, $N'_{20} = N_{30}$. This correlation has been used in calculations based on SPI though it, however, is valid provided that a free falling hammer and 0.76 m height of fall is used.

- The method based on pressuremeter test results in accordance with Ménard as described by Baguelin et al (1978).

The results of the calculation of q_a and q_u for the different slabs are summarized in Tables I-IV. The calculated values vary widely with both the slab width and the method of calculation.

TABLE I. Allowable bearing pressure at Kolbyttemon by different calculation methods.

Plate size (m)	Allowable ground pressure, q_{all} (MPa)							
	Swe. Build Code	Swe. Road Adm. Code	Danish Code	Meyerhof, SPT/HfA		Meyerhof, CPT		Menard
				$\frac{q_{ult}}{3}$	$q^{(1)}$ s<25 mm	$\frac{q_{ult}}{3}$	$q^{(1)}$ s<25 mm	
0.55 x 0.65	0.08	0.11	0.16	0.11	0.14	0.26	0.38	1.03
1.10 x 1.30	0.17	0.22	0.33	0.22	0.14	0.52	0.38	1.03
1.60 x 1.80	0.27	0.32	0.52	0.32	0.13	0.76	0.32	1.03
2.30 x 2.50	0.40	0.46	0.67	0.46	0.12	1.09	0.29	1.03
2.00 x 3.00	0.30	0.27	0.23	0.28	0.11	0.66	0.30	0.78

TABLE II. Ultimate bearing pressure at Kolbyttemon by different calculation methods.

Plate size (m)	Ultimate ground pressure, q_{ult} (MPa)			
	Danish Code	Meyerhof SPT/HfA	Meyerhof CPT	Menard
0.55 x 0.65	0.34	0.33	0.78	3.09
1.10 x 1.30	0.68	0.67	1.56	3.09
1.60 x 1.80	0.94	0.97	2.28	3.09
2.30 x 2.50	1.41	1.39	3.27	3.09
2.00 x 3.00	0.53	0.84	1.98	2.33

1) According to Meyerhof (1965) these allowable bearing pressures can be increased by 50% without settlements exceeding 25 mm.

TABLE III. Allowable bearing pressure at Fittja by different calculation methods.

Plate size (m)	Allowable ground pressure, q_{all} (MPa)						
	Swe. Build. Code	Swe. Road Adm. Code	Meyerhof, SPT/HfA ²⁾		Meyerhof, CPT ²⁾		Menard
			$\frac{q_{ult}}{3}$	$q^{(1)}$ s < 25 mm	$\frac{q_{ult}}{3}$	$q^{(1)}$ s < 25 mm	
0.55 x 0.65	0.02	0.04	0.02	0.02	0.04	0.05	0.16
1.10 x 1.30	0.04	0.09	0.04	0.02	0.07	0.05	0.16
1.60 x 1.80	0.07	0.13	0.05	0.02	0.10	0.04	0.16
2.30 x 2.50	0.11	0.18	0.08	0.02	0.14	0.04	0.18
2.40 x 3.60	0.10	0.13	0.06	0.02	0.12	0.04	0.16
2.80 x 4.20	0.11	0.13	0.07	0.02	0.14	0.04	0.16

TABLE IV. Ultimate bearing pressure at Fittja by different calculation methods.

Plate size (m)	Ultimate ground pressure, q_{ult} (MPa)		
	Meyerhof SPT/HfA	Meyerhof CPT	Menard
0.55 x 0.65	0.06	0.10	0.47
1.10 x 1.30	0.11	0.20	0.47
1.60 x 1.80	0.15	0.30	0.47
2.30 x 2.50	0.22	0.43	0.53
2.40 x 3.60	0.18	0.35	0.47
2.80 x 4.20	0.22	0.42	0.47

1) According to Meyerhof (1965) these allowable bearing pressures can be increased by 50% without settlements exceeding 25 mm.

2) According to Meyerhof (1956) the bearing capacity has been reduced for the effect of water table within the depth of 1.5 B below base level. For plate 2.80 x 4.20 the reduction factor has been 0.56 and for the rest of the plates 0.50.

SETTLEMENT CALCULATIONS

The settlement calculations were performed according to different methods described in literature. Methods based on both static and dynamic penetration tests have been used, as well as that proposed by Ménard for pressuremeter test results. Oedometer test results were also used for material from Kolbyttemon. Calculation methods used:

- For CPT results: De Beer (1965) and Schmertmann (1978).
- For SPT and HfA test results: Schultze & Sherif (1973) and Parry (1977).
- For pressuremeter test results: Ménard (1965).
- For oedometer test results the compressibility was evaluated. The stress distribution was assessed using elastic theory.

Settlement calculations were performed at different bearing pressures for all slabs at both sites. In methods where the stress distribution can be considered the settlements were calculated for the stresses below the characteristic point. Mean soil pressures were applied elsewhere.

The results of the settlement calculations in Fig. 5 show the settlement variation with slab width for the different methods when an allowable bearing pressure corresponding to the SNRA Code was applied.

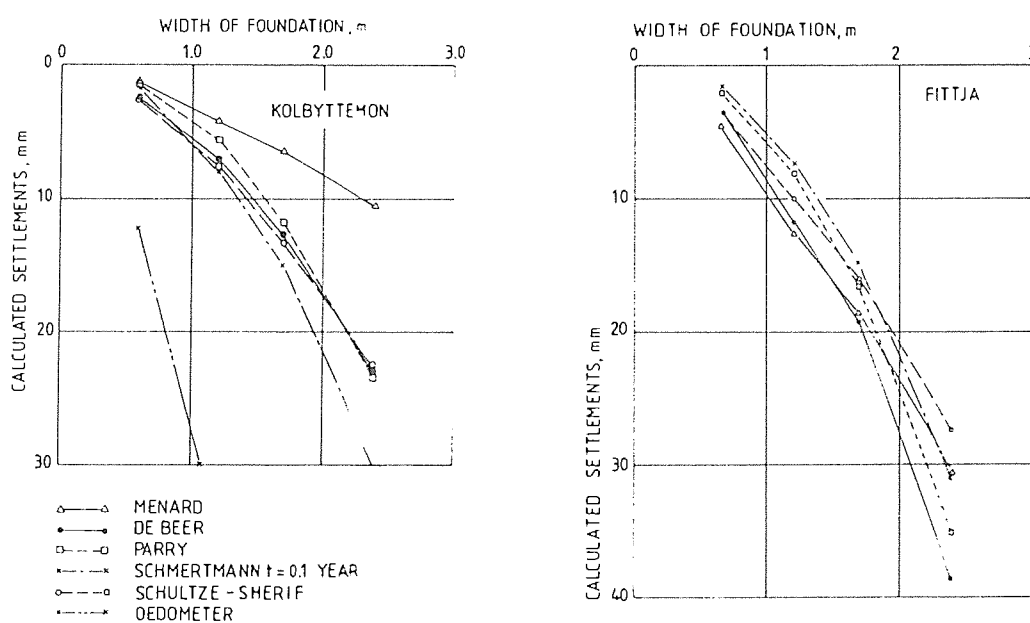


Fig. 5 Calculated settlements according to different methods at q_a from the SNRA Code.

The values for Kolbyttemon show a rather good agreement between the four methods based on penetration tests. The Ménard method gives, however, only about half of these values and the oedometer-based method gives much higher values. For Fittja there is quite good agreement between all the five methods used. In this comparison it should be noted that values according to Schmertmann are valid at 0.1 years and the others after a long time (>10 years).

RESULTS OF PLATE LOAD TESTS ON SLABS

The results of the load tests are summarized in Fig. 6 for all slabs at both sites.

From the test results it is very difficult to evaluate an ultimate bearing pressure, q_u , partly because of the low range of pressures applied and partly due to the continuous bending of the curves.

For the smallest slabs (0.55 x 0.65 and 1.10 x 1.30 m) at Fittja, however, a settlement corresponding to about 10% of the width of the footings was reached. This deformation can be used as a failure criterion. Thus, q_u for these slabs were 610 and 260 kPa respectively. The lowest value, paradoxically, was obtained for the larger of the two slabs. Additional soil investigation revealed a clay lens between 0.2-0.7 m below the slab.

At Kolbyttemon a settlement of about 7% of the width was reached when testing the smallest slab. The maximum load applied was then 1400 kPa. Extrapolating the load-settlement curve for this test gives a failure load of about 1600 kPa at 10% deformation.

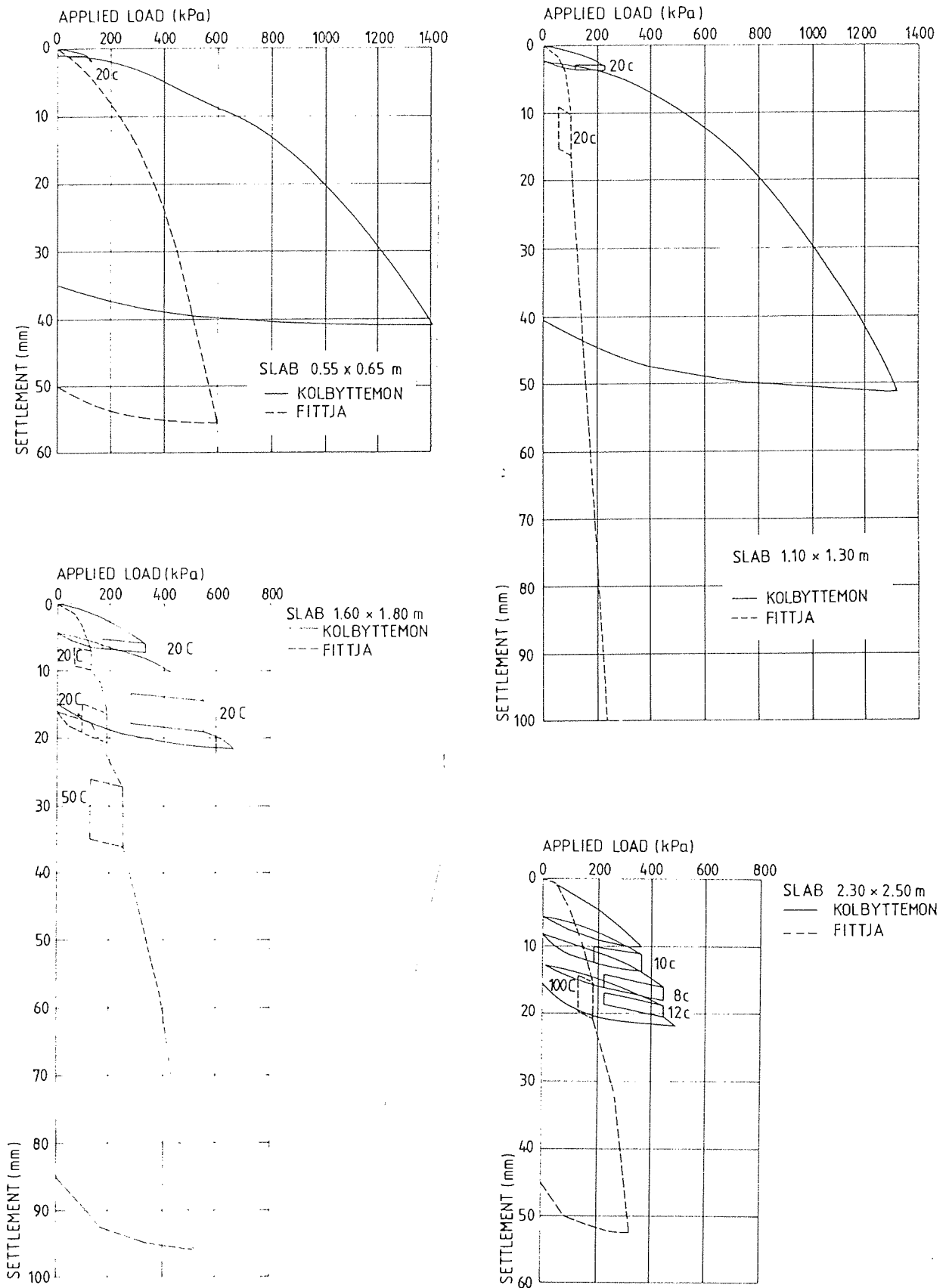


Fig. 6 Load-settlement curves from tests at Kolbytte-mon and Fittja (20 c indicates 20 loading/unloading cycles).

The bearing capacity can also be studied by the yield stress, q_y . This stress has been evaluated from some of the load-settlement graphs and has then been defined as the stress at the intersection of two tangents to the load-settlement curve; one through the elastic part of the curve and one through the plastic part.

The yield stress can also be evaluated from the creep curves achieved when testing the slabs with incremental increase of the load. The creep curve gives the relationship between the applied load and the additional settlement during a specific time interval (creep) when keeping the load constant. For loads higher than the yield stress there is a considerable increase in the creep. The yield stress has been defined as the stress at which the creep curve has the least radius of curvature.

From the tests at Kolbytteimon the yield stress cannot be evaluated in the latter way for any of the slabs. For each one the yield stress is obviously higher than the maximum load in the opening ML-tests. At Fittja the yield stress can be evaluated only for the two smallest slabs due to the loading program. The measured creep for slab 0.55x0.65 m is shown in Fig. 7.

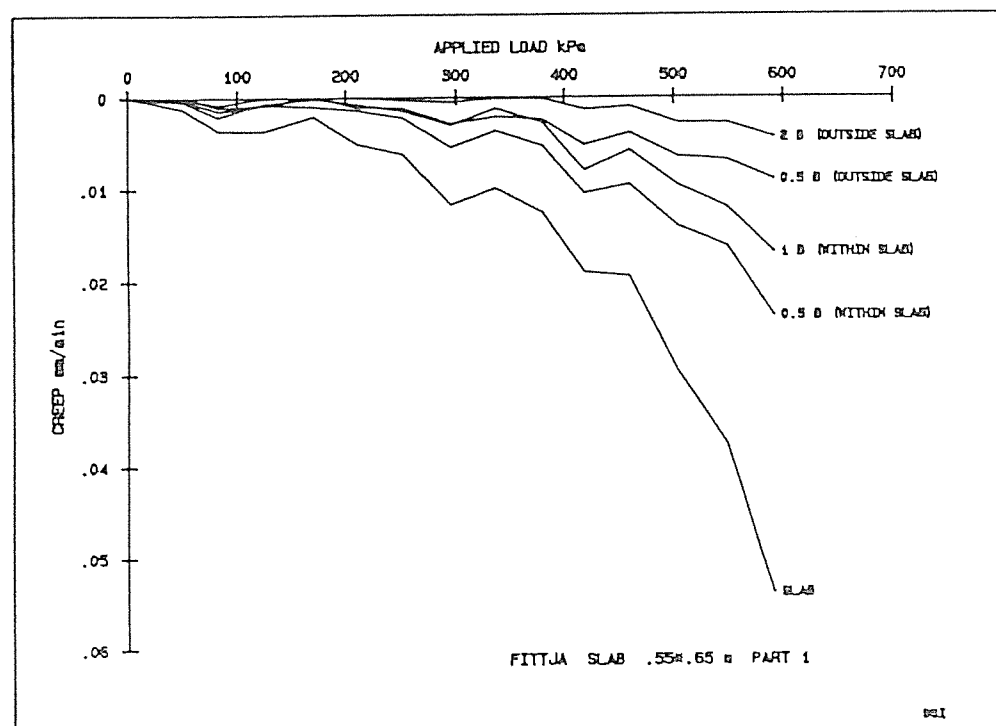


Fig. 7. Creep, 30-60 min, vs load for slab 0.55x0.65 m and different levels beneath the slab, Fittja.

The evaluated values q_y determined in one of the above described ways for the smaller slabs at the two sites are shown in Table V. It is surprising that the yield stress does not increase with slab size at Kolbytte-mon.

TABLE V. Yield stresses, q_y , for the slabs at the two sites.

Slab size (m)	Yield stress, MPa	
	Kolbytte-mon	Fittja
0.55 x 0.65	0.83	0.45
1.10 x 1.30	0.72	(excl. due to clay)

The settlements of the abutments at both sites were followed during a period of about 40 days after the construction of the counterweight. The initial settlements after the construction were 7.2 and 9.3 mm respectively at Kolbytte-mon. At Fittja the initial settlement was 34.9 mm for the 2.4 x 3.6 m abutment and 26.3 mm for the 2.8 x 4.2 m abutment. The relative settlement increase during the 40 days of observations is shown in Figs. 8-9. At Kolbytte-mon the relative settlement increased almost linearly with the square root of the time, to about 50% after 40 days.

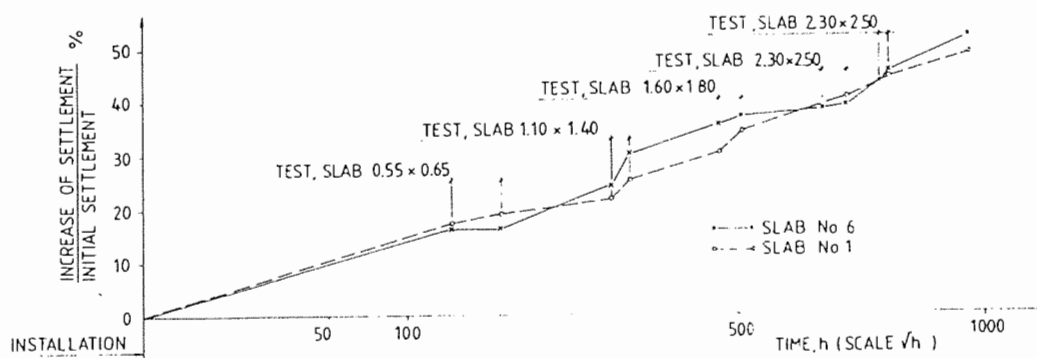


Fig. 8 Abutment settlement increase at Kolbytte-mon.

At Fittja the relative increase in the settlements at the beginning did not follow a linear relationship. At the end of the observation period pile driving nearby increased the settlements considerably. The relative increase was about 40 and 60% during the observation period, excluding the effect of pile driving.

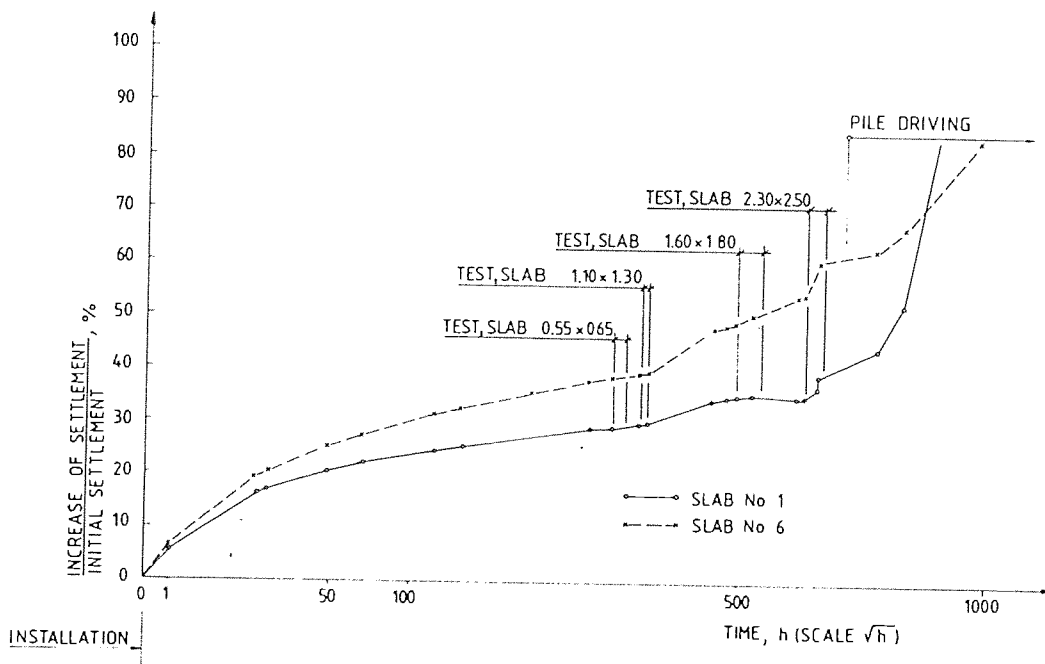


Fig. 9 Abutments settlement increase at Fittja.

The settlement distribution versus depth for the two sites is shown in Fig.10. The dots in the graph show the average values for each depth. The results indicate that about 80% of the settlement is due to compression in the upper $1 \times B$ thick layer and that only 10% comes from depths below $1.5 \times B$.

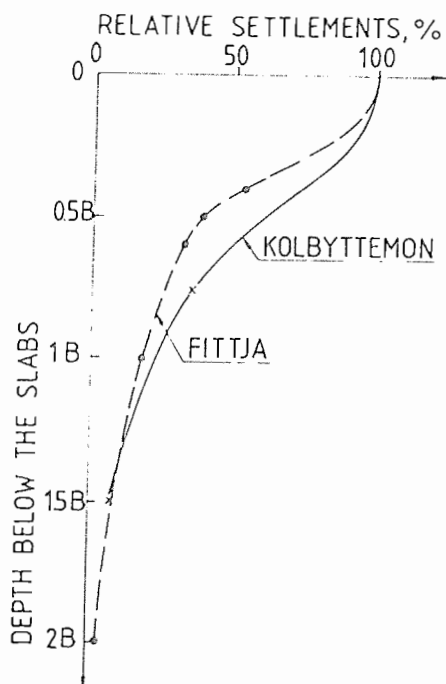


Fig. 10 Settlement distribution below the slabs at Kolbytteimon and Fittja.

COMPARISONS BETWEEN CALCULATED AND MEASURED VALUES FOR BEARING CAPACITY AND SETTLEMENTS

In the following comparisons slab 1.10 x 1.30 m at Fittja has been omitted because of the clay lens under it. Thus, the only ultimate bearing capacity value obtained from the load tests is for the smallest slab (0.55 x 0.65). At Fittja this value is 0.61 MPa and at Kolbytteimon 1.6 MPa approximately.

The first value can be compared to those obtained using the Meyerhof and Ménard methods, 0.06 - 0.47 MPa, Table IV. Consequently, these calculations underestimate the bearing capacity by a factor of 1.3-10.2. The Kolbytteimon value is to be compared to those in table II, 0.33-3.09 MPa. The highest value by Ménard is an overestimation by about 2 while the other methods underestimate the bearing capacity by a factor of 2-5.

A comparison between the measured q_y , Table V and the calculated q_u , from the two sites, Tables II and IV shows that the yield stress can be as high as 7.5 and as low as 0.2 times the calculated ultimate bearing pressure.

It seems that the calculations gave too low ultimate bearing capacity values for the smallest slabs except the Ménard method which considerably overestimates the bearing capacity of the dense sand at Kolbytteimon.

For the 1.1 x 1.3 m slab at Kolbytteimon, with dense material, the Meyerhof method based on SPT and the Danish formula gave bearing capacities equal to the yield stress while the Meyerhof method based on CPT gave a bearing capacity about twice the yield stress. On the other hand the latter method at this site gave a bearing capacity for the smallest slab equal to the yield stress.

At Fittja the Ménard method gave a bearing capacity equal to the yield stress for the smallest slab for which the methods by Meyerhof considerably underestimate this value.

A comparison between calculated and measured settlements for the 1.6 x 1.8 m slab is shown in Fig. 11.

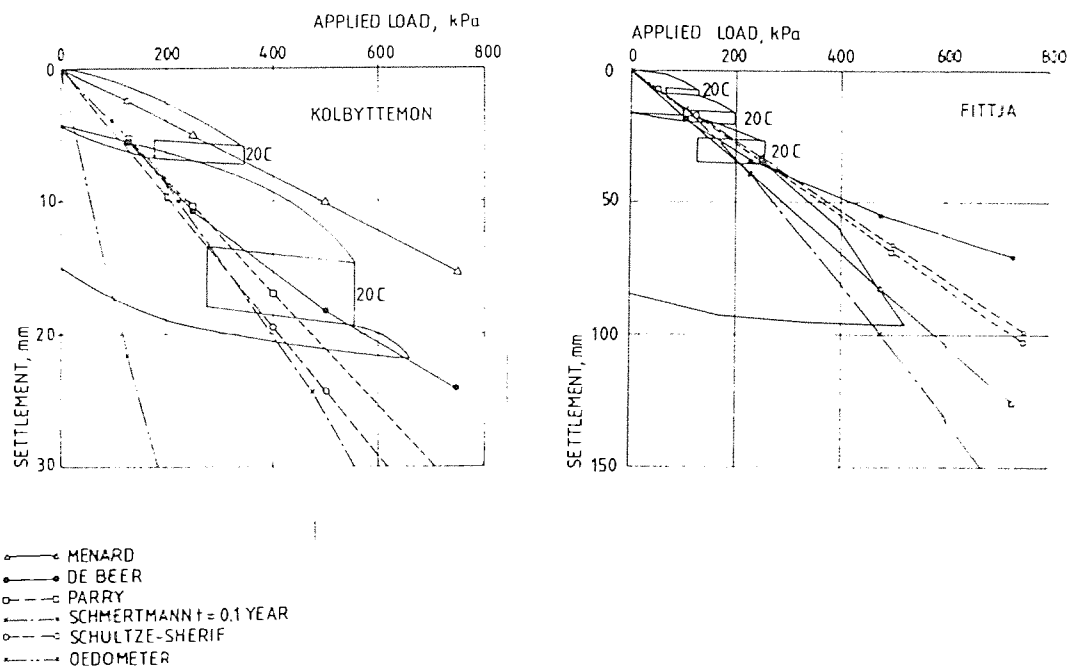


Fig. 11 Calculated and measured settlements for the 1.6 x 1.8 m slab at Kolbytteimon and Fittja.

At Kolbyttemon the result of the Ménard method agreed well with the initial parts of the load-settlement curves while methods based on penetration testing gave values corresponding to settlements after cyclic loading and unloading or after some time.

At Fittja calculated settlements are greater than those measured corresponding to initial load-settlement curves, 20 to 100 cycles at q_a according to the SNRA included. At bearing pressures $> 2 \times q_a$ the settlements of repeatedly loaded test slabs agree well with those calculated. Final settlements of the abutments correspond quite well to the calculated values if the effect of piling is omitted.

The methods used for calculating settlements give values valid after long-term loading (~10 years), excluding Schmertmann's. As the load tests were performed within 24 h the measured settlements must be recalculated in some way to be fully comparable with those calculated. Figs. 12a-12b show the ratio of measured/calculated settlement versus width of slab at q_a . The graphs show the ranges according to values before and after 20-100 cycles. Using the change in settlement measured on the abutments (as an average 50% increase during the first 40 days) will give ratios at q_a before cycles in accordance with graphs 12c-12d. In these comparisons it should be noted that values according to Schmertmann are valid at 0.1 years and the other still after a long time (10 years).

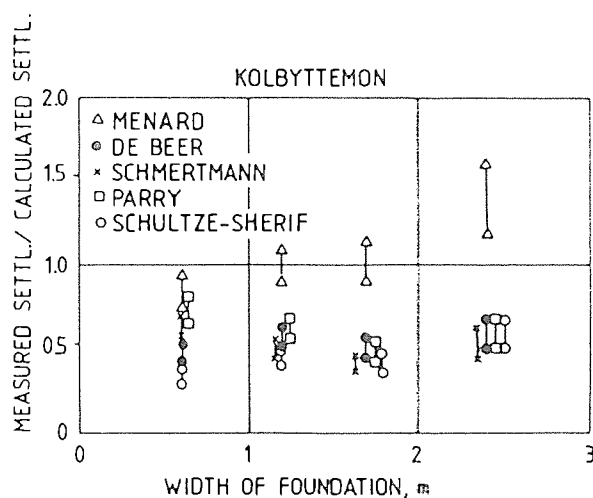


Fig. 12a

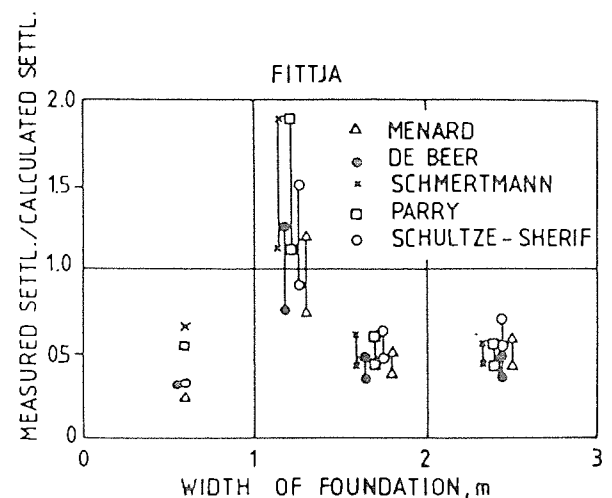


Fig. 12b

Fig 12a, 12b. Ratio measured settlement/calculated settlement before and after 20-100 cycles for tests at both sites.

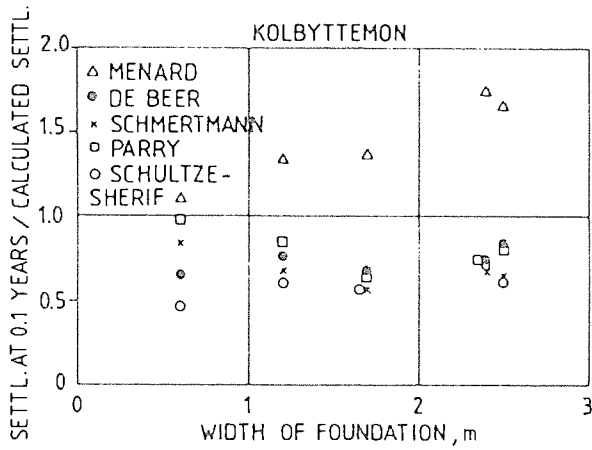


Fig. 12c

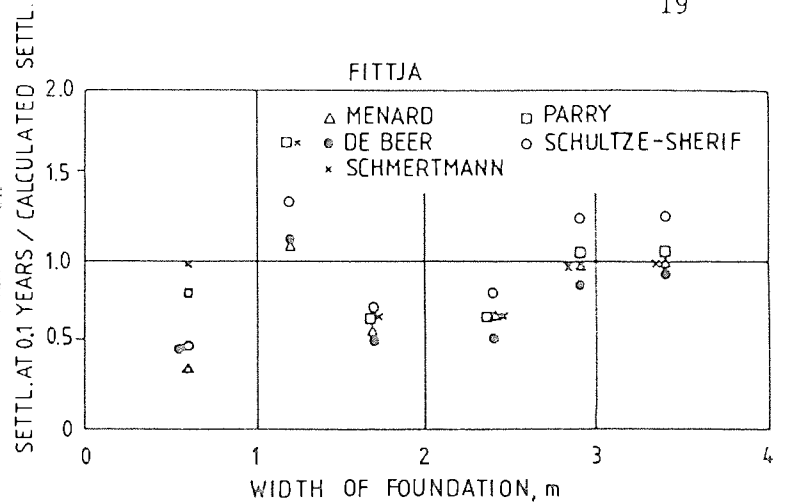


Fig. 12d

Fig. 12c, 12d. Ratio 'measured' settlement at 0.1 years/calculated settlement for tests at both sites.

For the Kolbytte-mon site (dense soil) it can be seen that the ratio based on penetration testing is independent of slab size. For the Fittja site (loose soil - slab 1.1 x 1.3 excluded) there is a good agreement between all methods. However, the ratio seems to be somewhat increasing with slab width.

According to Schmertmann the effect of time on settlements is given by the factor $c = 1 + 0.2 \log(\text{time}(\text{year})/0.1)$. This means an increase from 0.1 to 10 years of 40%. Assuming the slab settlements grow in this way the 10-year ratios will be as given in Figs. 13a-13b.

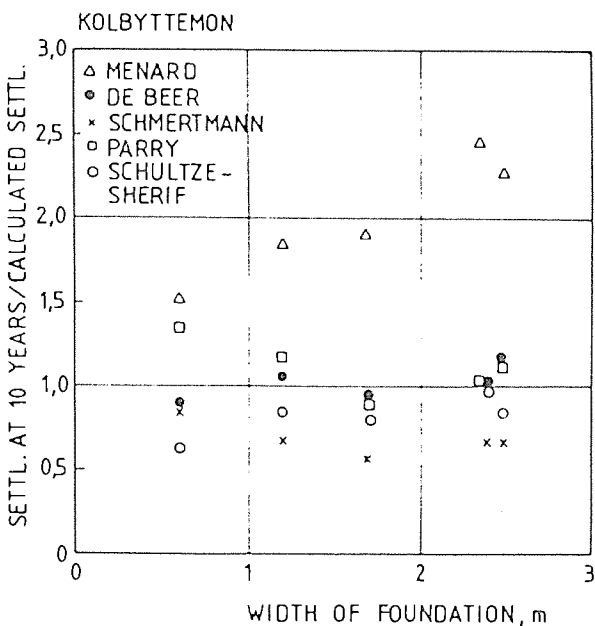


Fig. 13a

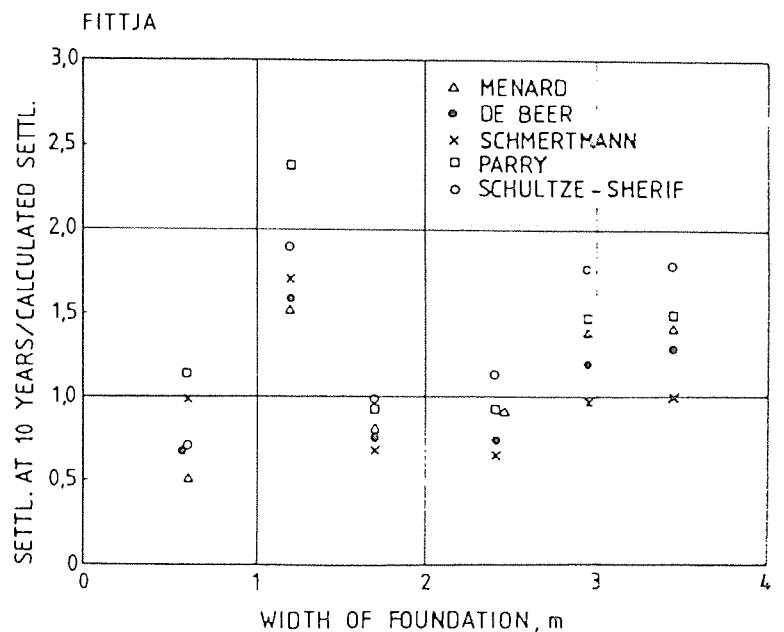


Fig. 13b

Fig. 13a, 13b. Ratio settlement at 10 years/calculated settlement at 10 years for tests at both sites.

Under the above assumptions the methods of De Beer, Parry and Schulze-Sherif in mean give results of high accuracy for the long-term settlements at Kolbyttemon. On the Fittja site these methods as well as Ménard's in mean will do the same for the test slabs. On both sites Schmertmann's method overestimate settlements by about 30%, the abutments at Fittja excluded for which this method gives results of high accuracy.

De Beer (1965) states that settlements calculated with the formula used here will give values on average twice as high as those observed, that is the long-term ratio should be 0.5. The result above is in contradiction to this.

CONCLUSIONS

The test series show that the settlements achieved are strongly affected by the loading programme. The final settlements at a particular bearing pressure are highly dependent on the number of load cycles, the loading amplitude and the loading time.

The test results also show that the methods used for calculation of long-term settlements at bearing pressures less than twice the allowable pressure (according to the Swedish National Road Administration Code) in general agree quite well. The magnitude of the long-term settlements of a foundation can be calculated with reasonable accuracy for engineering assessments using these methods.

The settlement distribution versus depth indicates that about 80% of the total settlements occur within the upper $1 \times B_m$ layer and less than 5% below $2 \times B_m$.

The methods used for calculating the bearing capacities give results with a rather large scatter. Evaluated yield stresses seem to be rather constant, that is independent of the slab width which is in accordance with the ultimate bearing pressures derived by the Menard method, though these values are from one to four times the evaluated ultimate bearing pressures.

The other methods used show an increasing ultimate bearing pressure with slab width. In total the evaluated yield stresses are in the range of 0.2-7.5 times the calculated ultimate bearing pressures. If the bearing capacity is evaluated as the load giving a settlement of 10% of the slab width the measured ultimate bearing pressures are in the range of 0.5-10 times the calculated values.

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