SECONDARY SETTLEMENTS OF BUILDINGS IN DRAMMEN, NORWAY

TASSEMENTS SECONDAIRES DE BATIMENTS A DRAMMEN, NORVEGE

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SYNOPSIS The paper presents the results of a settlement study of three buildings in Drammen. The buildings are founded on deep deposits of soft clay and have experienced large settlements in spite of the fact that the clay appears to be lightly overconsolidated. The settlements are analysed according to a concept presented by Bjerrum (1967), which explains the apparent overconsolidation to be caused by delayed (secondary) consolidation of the clay after sedimentation. Settlements estimated on this basis are in good agreement with the observations, both with respect to final settlements and the rate of settlement. The validity of the concept is thereby proved.

INTRODUCTION

The city of Drammen, 40 km south-west of Oslo, is well-known for its poor soil conditions. Since the Norwegian Geotechnical Institute was founded, it has made a comprehensive study of the properties of the clays and the settlements of buildings in the city. Today, reliable and detailed settlement observations exist for a great number of buildings, most of them 10-20 years old.

In his Rankine lecture, Bjerrum (1967) briefly described the settlements of a number of buildings in Drammen for which the settlement behaviour differed significantly from the settlements that can be predicted from a conventional analysis. The discrepancy was explained by introduction of the concept of "delayed consolidation". According to this concept, an apparent preconsolidation pressure, p_c, exceeding the vertical effective overburden pressure p'_o will develop as a clay experiences delayed (or secondary) consolidation. This effect may be detected by consolidation tests, but since p_c is time dependent and reduces with time, the effect cannot be fully relied upon when designing building foundations. The clay behaves as overconsolidated for small additional loads, but when the stresses approach and exceed p_c, then the behaviour approaches that of a normally consolidated clay.

In the present paper is presented the result of a study of the buildings for which the most complete settlement observations are available and the observed settlements are then used to test the validity of the concept of delayed consolidation.

SOIL CONDITIONS

The geologic history of the area considered is described by Bjerrum (1967). The buildings are situated in a low-lying area, about 2 m above mean sea level. In fig. 1 the 2 m contour is shown from a map prepared in 1912, at a time when the area was very little developed. Since the general land upheaval in the area is at present about 30 cm per century, the area below the 2 m contour has probably risen above sea level 700-1000 years ago.

Fig. 2a shows the location of the buildings and borings. Except for a small variation in the thickness of an upper sand layer the soil conditions in the area were found extremely uniform. Boring 1 was made for investigation of the consolidation characteristics of the clay and was situated about 50 m south of Konnerudgate 16. It made use of the NGI 95 mm thin-walled piston-sampler which had already been proved to give excellent undisturbed samples of soft clay. Utmost care was taken to avoid sample disturbance during transportation, and the samples were tested within a few weeks after arriving in the laboratory. The samples were tested by Clausen (1966).

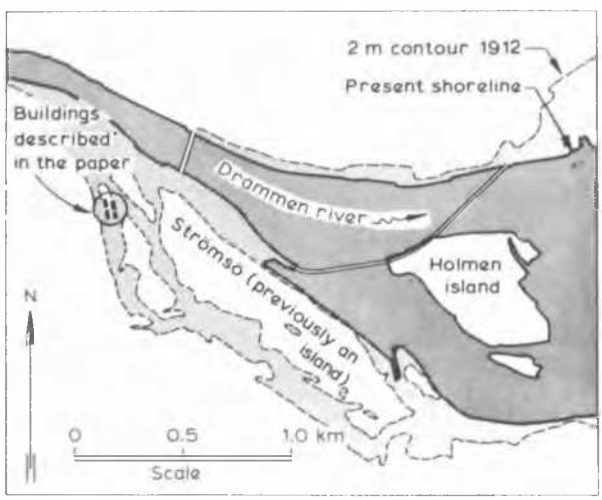


Fig. 1. Map of a part of Drammen showing present and old shoreline.

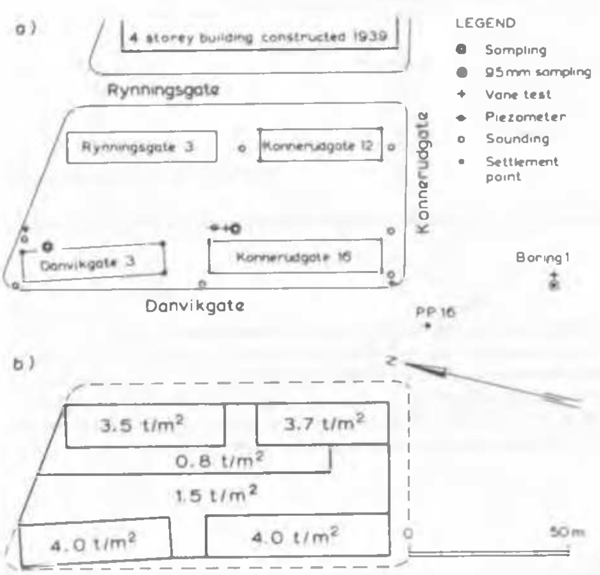


Fig. 2. a) Location of buildings and borings.
b) Net additional loads assumed in settlement computation.

A soil profile from boring 1 is shown in fig. 3. Under a thin layer of top soil there is a 2 m thick layer of uniformly graded fine sand, containing very little organic matter. Below 2.5 m depth there are various clay layers. The upper 2.5 m is a soft silty clay with a water content of 40-45%. The undrained shear strength as measured by vane tests is slightly below 2 t/m^2 and the sensitivity about 10.

Below the silty clay is a 5.5 m thick layer of plastic clay which has been shown to be responsible for a major part of the settlements of the buildings. It appears as a dark grey homogeneous soft clay with water content 45-55%. The maximum water content is typically found in the middle of the layer. The content of clay size particles is 40-50%, the liquid limit about 60 and the plastic limit 30, giving a plasticity index of 30. The water content is slightly below the liquid limit. The undrained shear strength of the plastic clay increases from 2 t/m^2 at the top of the layer to 3 t/m^2 at the bottom. The sensitivity is 7-10. The clay is slightly organic, the content of humus being about 1% of the dry weight.

At about 10.5 m depth there is a well-defined transition zone from the plastic clay to an underlying lean clay. The transition zone contains numerous shells, small pebbles and occationally some pockets or thin layers of sand are found. With respect to consolidation of the deposit, the transition zone is not believed to provide drainage possibilities. This is substantiated by field observation of pore pressure dissipation in the same area.

The lean clay extends from 10.5 m to at least 36 m depth, and soundings to about 60 m have not reached rock. The lean clay has a water content of 30-35%

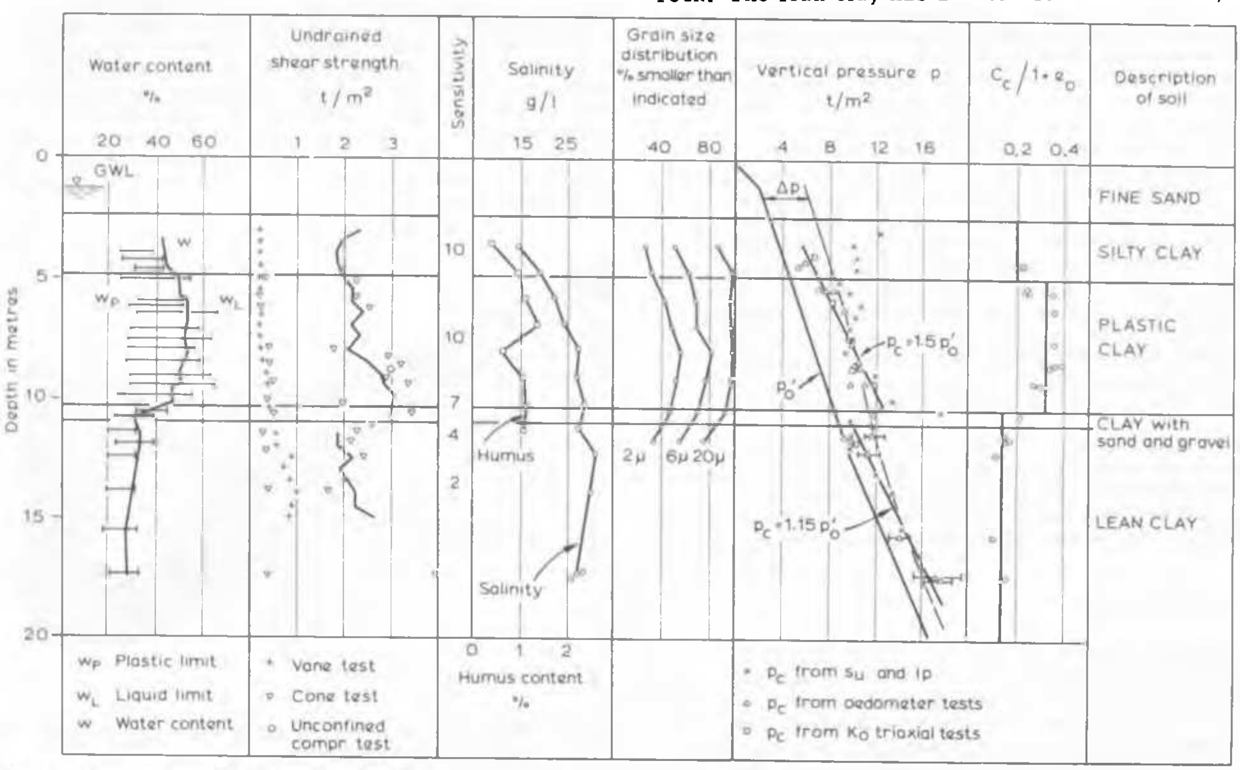


Fig. 3. Boring profile from boring 1.

corresponding approximately to the liquid limit, and the plasticity index is 10-15. The undrained shear strength drops sharply from 3 t/m^2 to about 2 t/m^2 across the transition zone and in the lean clay increases slightly with depth. Other borings show that at 20 m the shear strength is about 2.4 t/m^2 only. Below 25 m the clay becomes more silty and varved. Occasional silt seams are also observed at more shallow depths.

The studies included determination of the salt content, see fig. 3. Below 10 m depth the salinity is about 30 g/l, while det clay is slightly leached at shallower depths. This reduction in salt content has not caused any change in the clay properties.

Piezometers were installed at different depths. These indicate a small artesian pore pressure of about 1 m of water at 20 m below the ground water level which is about 1 m below natural ground elevation. The seasonal variation in the ground water level in the sand is about $\pm$ 0.3 m, while the pore pressure in the clay only varies by about $\pm$0.1 m. The long-term trend in pore pressure at a given depth is a small but steady decrease as the land rises with respect to the mean sea level.

The soil conditions at Konnerudgate 16 and Danvikgate 3 are identical to those described above for boring 1, except that the thickness of the plastic clay is 5 m. At Konnerudgate 12 the thickness of the sand layer is 4 m, but otherwise the soil conditions are as described above.

CONSOLIDATION CHARACTERISTICS

The consolidation characteristics of the clays were investigated by means of oedometer tests and triaxial K_o-consolidation tests. The specimens were mounted in the testing devices with utmost care to minimize sample disturbance. The equipment and procedure used for triaxial specimens is described by Landva (1964). The specimens were 10 cm high and 5 cm in diameter. They were first consolidated under an all-round pressure of 0.25 kg/cm^2 to avoid swelling when the negative pore pressures were released. Then the cell-pressure and the vertical pressure were increased in small increments so that the cross-sectional area of the specimen remained constant. In this way, the ratio between cell pressure and vertical pressure approached a constant value for pressures above p_c corresponding to the in situ K_o stress ratio which was found to be 0.45-0.55.

Oedometer specimens 8 cm diameter and 2 cm high were prepared using similar methods and were tested in double draining oedometers using Teflon-coated fixed rings. The duration of each load increment was usually 24 hours, and the load increment ratio $\Delta p/p$ was usually 0.2, 0.5 or 1.0.

The settlement analysis is based on a total of 22 oedometer tests and 4 triaxial tests. Typical settle-

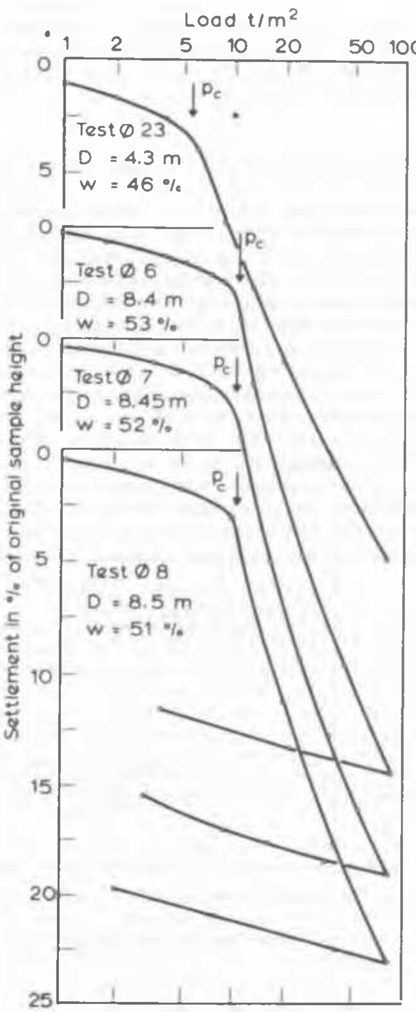


Fig. 4. Settlement-log p curves for oedometer tests on silty and plastic clay (fig. 4a, left) and on lean clay (fig. 4b, below)

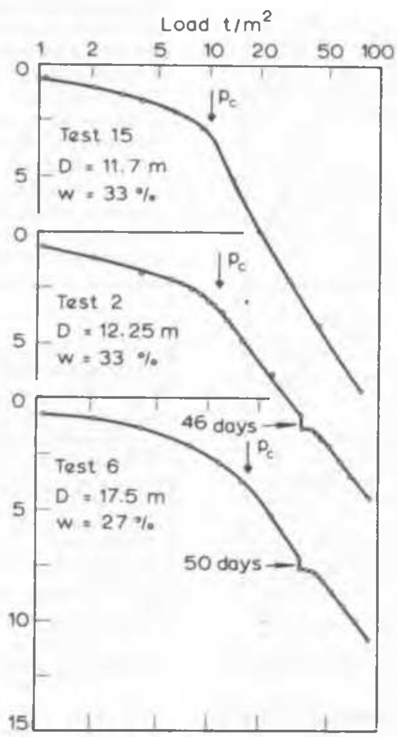
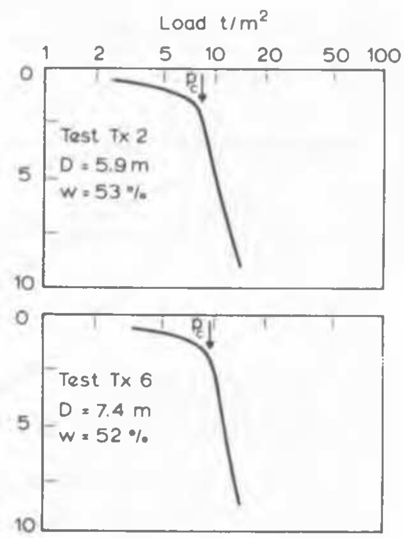


Fig. 5 (below). Settlement-log p curves for K_o-consolidated triaxial tests.



ment - log p curves for 4 oedometer tests on the silty and plastic clay and 3 tests on the lean clay are given in fig. 4a and b. Similar curves for 2 triaxial tests on the plastic clay are given in fig. 5. Tests on the silty and the plastic clay show pronounced 'breaks' at the preconsolidation pressure p_c which is indicated with an arrow. On the lean clay, the arrow indicates the most probable value of p_c, since the break for this clay is less clearly defined. The values of p_c for all tests are given in fig. 3. In the silty and the plastic clay, p_c is about 1.5 times the effective overburden pressure p_o' while in the lean clay $p_c \approx 1.15\ p_o'$. * The same diagram also shows the values of p_c estimated from the undrained shear strength and the plasticity index, (Bjerrum 1967). The correlation is good except near the ground surface where the method appears to overestimate p_c.

* Fig. 3 is slightly different from the corresponding fig. 19 in Bjerrum's paper (1967). Some rapid tests are disregarded and new tests added. Field tests have shown that the silty clay is probably not significantly weathered as suggested in Bjerrum's fig. 19. As a consequence, p_c is slightly altered and thereby also the ratio $\Delta p/(p_c - p_o')$ which will later be given for each building.

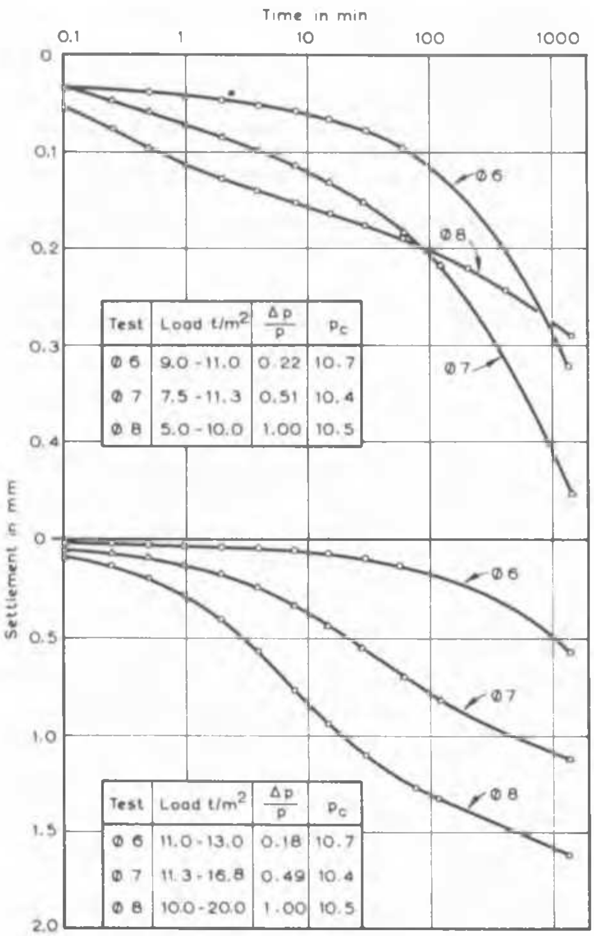


Fig. 6. Time settlement curves for oedometer tests on plastic clay.

The consolidation curves from the triaxial tests show smaller deformations below p_c and even sharper breaks at p_c than the oedometer curves, thus indicating less sample disturbance. However, the observed values of p_c and of $C_c/(1+e_o)$ do not differ significantly from those found by oedometer tests.

The compressibility of the clay expressed as $C_c/(1+e_o)$, where C_c is the compression index and e_o original void ratio, is in clays of this type determined for the steep part of the settlement - log p curves at pressures just above p_c. This slope corresponds to the pressure range used later in the settlement analysis. Values of $C_c/(1+e_o)$ from the oedometer tests are shown in fig. 3. As observed the plastic clay is very compressible and the average value of $C_c/(1+e_o)$ is 0.32.

The shape of the time-settlement curve is similar to previous test results, for instance as reported by Leonards and Girault (1961). Fig. 6 shows typical examples of time-settlement curves for load increments at pressures close to p_c. These tests were made on samples from 8.4 - 8.5 m depth. The consolidation coefficient c_v increased greatly as the loads exceeded p_c by larger amounts. An average value of c_v was computed for the tests on silty and plastic clay. This is based on load increments just above p_c, most of them with $\Delta p/p = 1.0$, and results in a value of $c_v = 7.8 \cdot 10^{-8}\ m^2/s$.

DESIGN AND PERFORMANCE OF BUILDINGS

The buildings considered are four four-storey apartment buildings constructed 1949-1956, see table I, and their location is shown in fig. 2a. In fig. 2b the net additional loads and the weight of fill material used to raise the areas between the buildings is given. For one of the buildings shown in fig. 2, Rynningsgate 3, no settlement observations are made. This building was constructed 1952-53 and its weight is included in the stress computations for the other buildings. No loads other than those shown in fig. 2b have influenced the settlements. The buildings are all 10 m wide. Konnerudgate 12 is 39 m long, Konnerudgate 16 55 m long and Danvikgate 3 45 m long, and the buildings are of similar design. The foundation raft is 30-35 cm thick and extends 1.5 m

Building	Konne-rudgt. 12	Konne-rudgt. 16	Danvik-gate 3
Excavation started	15/9-55	1/12-53	autumn-49
Excavation completed	1/10-55	1/1 -54	autumn-49
Construction of building started	1/3 -56	1/1 -54	autumn-49
50% of net load applied	1/7 -56	1/5 -54	≈1/7-50
Full net load applied	1/10-56	1/11-54	Feb.-51
Total building weight*)	5.45 t/m²	5.45 t/m²	6.45 t/m²
Weight of excavation	1.75 "	1.45 "	2.45 "
Net additional load	3.7 "	4.0 "	4.0 "

*) Including 25% of the design live load.

Table I: Process of construction and building loads.

beyond the walls on all sides of the building. The basement, transverse walls and the floors are constructed in reinforced concrete. External longitudinal walls are of lightweight concrete and internal walls are brick.

Table I gives values of the average pressure on the ground, computed for the area of the foundation raft, and including 25% of the design live load. Unloading due to excavation has been calculated from working drawings, elevations of the ground before excavation etc. The actual net additional loads are believed to lie within ± 10% of the values stated, and are probably most accurately estimated for Konnerudgate 16. Danvikgate 3 has a double basement floor for drainage purposes, and this explains the higher weight of this building.

Bolts for measuring settlements were installed at the corners in Konnerudgate 12 and 16 during construction. On Danvikgate 3, the settlements of a pronounced cornice have been observed. The settlements are tabulated in table II and are plotted to a logaritmic time scale in fig. 7. Zero time has been taken as the time when 50% of the net load was applied, see table I.

The general ground subsidence in the area can be estimated from the settlement of the surveying reference point PP 16, see fig. 2a. In the period 1911 to 1959 the average, annual settlement was 1.8 mm/year.

Settlements before the first observation was made, have to be assumed. At Konnerudgate 12 observations started before any net additional load had been applied. Neglecting the small net heave or settlement at this time, the first reading is assumed to correspond to zero settlement. At Konnerudgate 16 the first observation was made when about 75% of the net additional load was applied. By backwards extrapolation of the settlement curve, settlements at the first observation were estimated to 2.2 cm. Judging from Konnerudgate 12, very small initial settlement had occurred. At Danvikgate 3, the first settlement observation was made when the building was ready for occupation. From the observed differential settlements and the settlement curve of Konnerudgate 16, the settlements have been estimated to 12.0 cm at the SE corner and 7.1 cm at the SW corner at the time of the first settlement observation.

At present the maximum settlement amounts to 28 cm for Konnerudgate 12, 45 cm for Konnerudgate 16, and 55 cm for Danvikgate 3. The differential settlements of the corners of the buildings amount to 26-32% of the maximum settlement, see Table II. The settlements of the basement floors at the centers of the buildings were determined in 1965 using spirit levelling and were found in each case to be about 3 cm more than the average settlement of the corners.

Large total settlements or differential settlements occurring as tilting do not cause damage to a structure. Damage is caused by deflection or distortion. In spite of the large differential settlements, small deflection or distortion of the buildings has occurred. The settlement of the center of the building with respect to the average of the corners can be used as a measure of the deflection. This difference is, as explained above, about

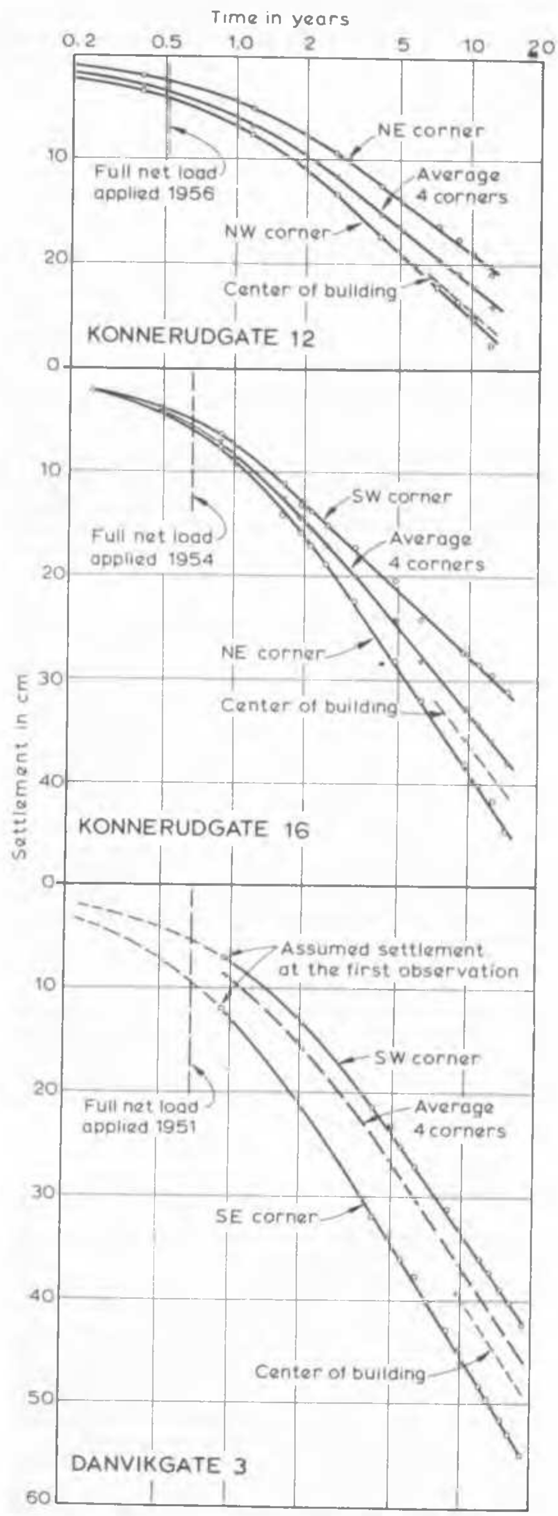


Fig. 7. Observed settlements for Konnerudgate 12, Konnerudgate 16 and Danvikgate 3.

Konnerudgate 12					Konnerudgate 16					Danvikgate 3				
Date	NE	NW	SE	SW	Date	NE	NW	SE	SW	Date	NE	NW	SE	SW
1/4 -56	(0)	(0)	(0)	(0)	1/8 -54	(2.2)	(2.2)	(2.2)	(2.2)	1/6 -51			(12.0)	(7.1)
15/6 -56	0.0	1.2	0.8	0.8	15/3 -55	7.6	7.6	7.6	6.4	1/9 -54			32.0	21.4
15/11-56	1.9	3.3	3.8	2.8	1/1 -56	14.2	-	-	11.2	1/5 -55			34.0	23.2
1/9 -57	5.0	7.6	7.0	7.5	1/4 -56	16.1	14.5	15.0	13.2	1/2 -56			36.1	25.0
15/3 -59	9.4	13.3	11.1	12.9	15/6 -56	17.2	15.4	16.0	13.7	1/12-56			38.0	26.9
9/9 -60	12.4	17.3	14.3	16.8	1/12-56	19.0	17.2	17.6	15.3	1/9 -57			40.3	28.6
25/10-63	16.2	22.0	17.4	21.0	1/9 -57	22.6	20.1	20.5	17.4	1/5 -59			43.0	31.2
19/ 5-65	17.6	23.7	18.9	22.7	15/3 -59	28.0	24.8	24.0	20.4	15/9 -62			48.4	36.2
30/10-66	18.8	25.3	20.0	24.0	9/9 -60	31.8	28.8	28.0	24.2	25/10-63			49.6	37.2
10/11-68	20.9	27.7	21.7	26.0	25/10-63	38.1	33.5	32.1	27.3	1/7 -65	45.4	35.1	51.6	39.0
					19/5 -65	40.2	35.2	33.8	28.7	30/10-66	46.4		53.0	40.2
					30/10-66	41.8	36.6	34.8	29.5	10/11-68	48.2		55.0	42.2
					10/11-68	44.5	38.7	37.0	31.2					

Table II : Observed settlements in cm of the corners of the buildings.

3 cm. The distortion may be evaluated from the difference in differential settlements across the building between the northern and southern ends. For Konnerudgate 16 this difference is zero, while for Konnerudgate 12 and Danvikgate 3 the difference is 2 - 2.5 cm.

Konnerudgate 16 and Danvikgate 3 have suffered some damage from the settlements. In Konnerudgate 16, 45° cracks developed in spite of the small deflection and distortion. The cracks developed in secondary building elements near the ends of the external longitudinal walls and were most pronounced in the upper storeys. Some cracks also developed in the connections between the longitudinal walls and the transversal reinforced concrete walls. The latter cracks may be due to causes other than settlements. The cracking demonstrates that lightweight concrete is very prone to cracking. The cracks were repaired in 1957 when the building was 3 years old. No damage has been reported later. In Danvikgate 3, no damage occurred to the building itself, but the sewage line located below the foundation raft has broken on two occasions. The line broke at the edge of the raft, obviously because of the large total settlements. The large differential settlements, particularly of Danvikgate 3 which leans 1 : 80, (0.7 degree), is of course noticed by the inhabitants, but the utility value of the building is not significantly reduced.

SETTLEMENT ANALYSIS

Settlement of buildings on clay may be divided into settlements caused by shear deformations, δ_s, and consolidation settlements accompanied by reductions in water content, δ_c. δ_s may be computed assuming the soil to be elastic and isotropic. For a safety factor against bearing capacity failure which in this case is about 3, the shear deformations will take place as the load is applied. For a given load, δ_s depends mainly on the value chosen for the modulus of elasticity, E. Bjerrum (1964) suggested $E = k \cdot s_u$, where k is a constant between 250 and 500 and s_u is the undrained shear strength. For a high safety factor, $k \approx 500$ and hence $E = 1100$ t/m^2. The corresponding settlement $\delta_s \approx 6$ cm. The settlement observations, particularly for Konnerudgate 12 indicates δ_s of the order 1 - 2 cm. The sand layer on the top of the clay has probably caused δ_s to be small, and has also reduced swelling and reconsolidation of the upper clay layer.

In the computation of consolidation settlements δ_c, the most important assumption to be made is to what extent settlements are influenced by the p_c-values observed in the consolidation tests. In fig. 3 the net additional stresses Δp under the center of Konnerudgate 16 are shown, together with the overburden pressure p_o' and the p_c-values. If p_c as found in consolidation tests was assumed to have a permanent effect on the settlements similar to the effect of a true overconsolidation, then small consolidation settlements would be computed. The inability of this interpretation to explain the large observed settlements led to the hypothesis that p_c is time-dependent. This hypothesis also explains how the clay obtained its p_c-value, as p_c was developed as a result of secondary consolidation which has occurred since the deposition of the soil (Bjerrum 1967).

Other explanations for the preconsolidation effect on this site have been considered but are rejected for the reasons described below :
Mechanical overconsolidation is not possible from geologic reasons. No material has even been eroded away and the pore pressure is slightly but steadily decreasing. Weathering is not probable because p_c-p_o' increases with depth within each soil layer and because p_c is abruptly reduced across the transition zone from the plastic to the lean clay. Furthermore, the observed settlements cannot be explained if p_c is caused by weathering. Cementation of clay particles would explain the observed p_c-values, but not the large observed settlements.

Provided p_c is caused by secondary consolidation as postulated by Bjerrum (1967), we can compute the sum of the primary and secondary settlements up to a time corresponding to the loading age of the sediment, which was assumed in this case to be 3000 years. Referring to fig. 8, the settlements will in each case correspond to the 3000 year curve. This curve is parallell to the 24 hours oedometer curve. The 3000 year settlements are thus computed by $C_c/(1+e_o)$ from the oedometer tests and assuming the clay to behave like a normally consolidated clay ($p_c = p_o'$). The values of $C_c/(1+e_o)$ used in the settlement computation are given by the solid line in fig. 3. Table III gives results of the settlement computation for Konnerudgate 16. Computed consolidation settlements after 3000 years are 60 cm for the center of this building, 45 cm for Konnerudgate 12 and 64 cm for Danvikgate 3. For comparison it can be mentioned that if it is assumed that p_c is not reduced with time then the computed consolidation settlements of for instance Konnerudgate 16 would be 16 cm only, compared with the observed 40 cm.

Clay layer	Depth m	$C_c/(1+e_o)$	Settlements in cm	Settlements in%
Silty	2.5 - 5	0.20	14.5	24
Plastic	5 - 10	0.32	28.5	47
Lean	10 - 35	0.15	17	29
Total			60	100

Table III : Computed total consolidation settlements for Konnerudgate 16.

No attempt has been made to compute differential settlements. The tilting of the buildings can be explained by the nonuniform distribution of net additional load, since fill has been placed on one side of the buildings only. For Konnerudgate 16 and Danvikgate 3, the depth of the excavation was larger along the western than the eastern wall. In addition, Konnerudgate 16 and Danvikgate 3 have each influenced the settlement of the other building.

The rate of settlement is usually computed under consideration of pore pressure dissipation only. According to the concept introduced earlier, a major part of the settlement is in this case assumed to be secondary, i.e. settlements under constant effective pressure. The rate of settlement is then governed by the compressibility characteristics of the grain skeleton, and not of the permeability and drainage conditions. This assumption has been substantiated by a recent load test on the same soil conditions and with similar load.

For a clay of this type, the rate of secondary settlement, ignoring any delay due to pore pressure dissipation, can be computed as follows from a diagram as shown in fig. 8.

In this idealized e-log p diagram the 24 hour reference curve represents the compressibility of the clay as for instance observed in oedometer tests. The 3000 year curve represents the relation between pressure and equilibrium void ratio after a time corresponding to the loading age of the deposit. The curves for intermediate times define the void ratio at given times after the load was applied. The curves in fig. 8 are assumed to represent an unique relationship between void ratio, effective stress and time.

For the special case of a building where $\Delta p = p_c - p_o'$, case 1 in fig. 8, the inter-sections between the vertical line at $p = p_c$ and the time curves define the settlement progress from 24 hours up to 3000 years. When $\Delta p > p_c - p_o'$, case 2, an instant compression, indicated by a dashed line in fig. 8, comes in addition to the secondary settlement. The rate of secondary compression is, however, the same for case 2 as for case 1.

In case 3 $\Delta p < p_c - p_o'$. Since the system of curves in fig. 8 is assumed to represent an unique relationship, the rate of settlement for case 3 immediately after loading will correspond to the rate of settlement for case 1 a certain time after the load has been applied. In fig. 8, this time is 10 years, and the time-settlement curve for case 3 from 0-2990 years after loading will therefore be identical to the time-settlement curve for case 1 from 10-3000 years.

Turning now to the actual case of the buildings in Drammen, the pore pressures set up during loading will complicate the idealized picture described above. Since the pore pressures are small and probably dissipated completely in a few years, they

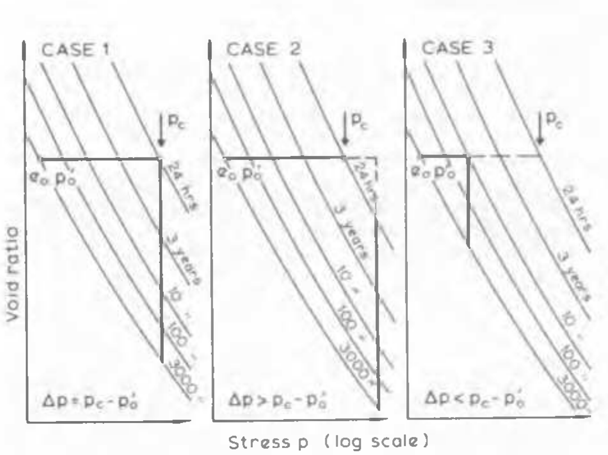


Fig. 8. Idealized relationship between void ratio, effective stress and time, illustrating principles of settlement calculation.

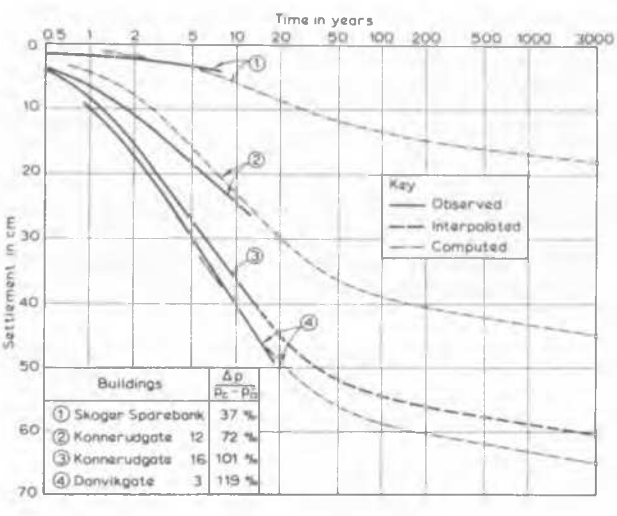


Fig. 9. Observed and computed settlements for the center of four buildings.

will influence only the first part of the settlement curve. The system of time curves had to be established from the observed and interpolated settlements for one of the buildings. The settlement curve for Konnerudgate 16, see fig. 9, is chosen for reference since for this building $\Delta p \approx p_c - p_o'$ in the middle of the plastic clay layer.*

According to the method described above the time settlement curve for Danvikgate 3 is simply found by displacing the curve for Konnerudgate 16 downwards. The settlement curve for Konnerudgate 12 is identical to the settlement curve for Konnerudgate 16, leaving out the first part of the curve up to 2.3 years after loading. The settlement curve for Skoger Sparebank has been included to demonstrate the method for a case where $\Delta p/(p_c-p_o')$ is small. The settlement curve is found as described for Konnerudgate 12, making small corrections for the thickness and the compressibility of the clay layers. The thickness of the sand layer is larger for Skoger Sparebank than for the other buildings, but otherwise the soil conditions are similar. Data about Skoger Sparebank are given by Bjerrum (1967).
A comparison of the settlement curves of the various buildings in fig. 9 indicates that the principles shown in diagram in fig. 8 closely describes the actual behaviour of the buildings with respect to secondary consolidation, even for a large range of variation in $\Delta p/(p_c-p_o')$.

CONCLUSIONS

1. Three 4-storey apartment buildings have settled 21-55 cm in 12-18 years. In spite of differential settlements of about 30% of the maximum settlements the tendency to deflection and distortion has been small. The settlements and damages which have occurred have not significantly reduced the utility value of the buildings.

2. Careful consolidation tests revealed that a 5 m thick layer of very compressible plastic clay underlying the area was slightly overconsolidated. Geologic considerations and the fact that large settlements must have occurred in this clay for loads not exceeding the preconsolidation pressure p_c led to the conclusion that the apparent overconsolidation is caused by secondary consolidation after sedimentation (Bjerrum 1967).

* The properties of all the clays involved have influenced the settlement curve, but the plastic clay provides a major contribution. However, all the clays are assumed to follow the general concept presented above.

3. A general relationship as illustrated in fig. 8 between void ratio, effective stress and time has been proved to exist for this type of clay. The application of this concept has in the first place led to an approximately correct prediction of final settlements of the buildings studied. In the second place the concept has also given a satisfactory account of the rate of settlements beyond the period of pore pressure observation.

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