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SWEDISH GEOTECHNICAL INSTITUTE

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Supplement to the ''Proceedings'' and ''Meddelanden'' of the Institute

Settlement Studies of Clay

1. Influence of Lateral Movement in Clay Upon Settlements in Some Test Areas

by Justus Osterman and Göte Lindskog

2. Consolidation Tests on Clay Subjected to Freezing and Thawing

by J. G. Stuart

STOCKHOLM 1964

Preface

Further research in the Institute's test fields since the issuing of its Proceedings No. 18 has resulted in, among other things, an unloading, to a certain extent, of one of the gravel embankments in the test area at Skå Edeby. The masses from the unloaded embankment have been used as a test road embonkment section and here deformation measurements of vertical and lateral movements, and pore pressure measurements have been performed. The new loading test is described and the problem of lateral soil movements below loaded areas is discussed in the first paper by J. Osterman and G. Lindskog. It is a reprint from the proceedings of the Wiesbaden Conference 1963 on problems of settlements and campressibility of soils.

In connection with the above investigations some of the measurements appear to have given results that could be considered to have been influenced by movements due to frost action. The supplementary, second paper by Dr Gordon Stuort, of the Queen's University of Belfast ond temporarily working ot the Institute, gives the results of some freezing ond thowing tests performed in the laboratory on soil samples during consolidation testing.

Stockholm, July, 1964

SWEDISH GEOTECHNICAL INSTITUTE

Influence of Lateral Movement in Clay Upon Settlements in Some Test Areas

By Justus Osterman and Göte Lindskog, Swedish Geotechnical Institute, Stockholm

Summary

When settlements of a soil area exposed to a load are discussed, the influence of lateral movements in the soil is often neglected. In the case of a relatively small loaded area on a soft soil these movements may however be important. Loading tests are reported where lateral displacement of the soil has been observed. Below a gravel embankment it was found that in the earlier stages the major part of the settlements was due to lateral movements of the soil at the boundaries of the loaded area.

Résumé

Lorsqu'on calcule le tassement d'une surface de chargement, on ne tient pas compte en général de l'influence des déplacements latéraux du sol. Dans le cas d'une extension limitée de charge reposant sur un sol de faible consistance, ces mouvements peuvent cependant devenir importants. Des essais de chargement sont présentés, dans lesquels on a pu observer des déplacements latéraux du sol. Sous un remblai de gravier, on a trouvé que la plupart des tassements pouvaient, dans les premiers temps, être reliés aux déplacements latéraux du sous-sol au bord de la surface de chargement.

Zusammenfassung

Bei der Erörterung von Setzungen einer Lastfläche wird häufig der Einfluß seitlicher Verschiebungen im Boden nicht berücksichtigt. Bei begrenzter Belastungsfläche können diese Bewegungen in weichem Boden jedoch von Bedeutung sein. Es wird über Belastungsversuche berichtet, bei denen seitliche Verschiebungen des Bodens beobachtet wurden. Am Beispiel einer Dammschüttung zeigte sich, daß in einem frühen Stadium die Setzungen zum großen Teil auf seitliche Verschiebungen im Boden am Rande der Lastfläche zurückzuführen waren.

Introduction

When a soil is consolidating under an overburden, the soil skeleton is exposed to stress causing settlement, which is delayed by the slow dissipation of pore water.

When a widespread load is applied to a level and homogeneous natural terrain the lateral deformations of the soil are prevented. This fundamental case has, as is well known, been discussed by several authors, and was first treated mathematically by Terzaghi.

Not much attencion is however paid to the more general case where lateral deformation may occur. At several bridges the senior author (Osterman, 1952) found that movements of the abutments were often due to lateral movements in the soil below the foundation level, especially in the case of clay. Similar findings are referred to in the literature (e.g. Tschebotarioff, 1958).

Review of Some Field Tests

In 1945 the Swedish Geotechnical Institute constructed a test field at Väsby, some 30 km north of Stockholm. Two areas 30 × 30 m were loaded with a 2.5 m fill of gravel (about 4.5 ton/sqm). The clay below one of the areas was drained with cardboard drains to 5 m depth with 0.7 m spacing, whereas the clay below the other area was undrained. After one year of loading, the fill on the drained area was decreased to 1.7 m (about 3.1 ton/sqm). The settlements were measured, but there were no arrangements for observations of lateral movements.

After unloading the pore water pressures still fluctuated a little and differed slightly from those of the surroundings, but the differences were negligible. The rate of the secondary settlements can be seen in fig. 1, which shows the compression of the 5 m drained layer mentioned.

In 1956 the question of the influence of lateral movements on the settlements was under discussion. At that time the compression of the drained layer of the field amounted to 16 per cent and should, theoretically, have been approximately equal to the loss in water content, the clay being almost saturated. The pore water pressures arising at loading were rather near to the intensity of the load applied. The actual decrease in water content was only in the region of 6 per cent, measured in various ways. It is, however, not appropriate to draw too definite conclusions about this result because of some possible shortcomings in the previous measurements. The clay is very homogeneous as regards strength and deformation properties, but it is

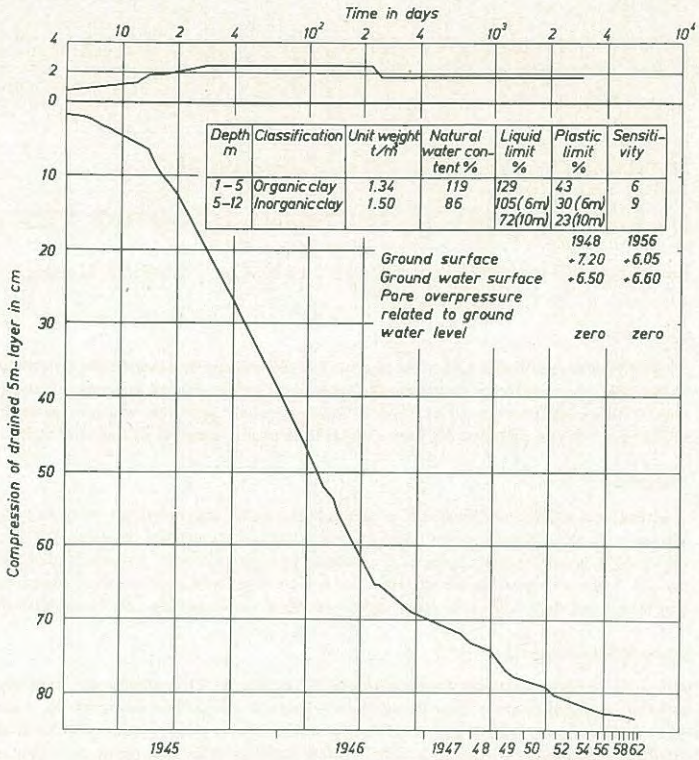


Fig. 1 Loading tests at Väsby. Compression of 5 m clay layer drained by cardboard drains, spacing 0.7 m. Size of test area 30 × 30 m

rather fissured. This latter fact can, together with some gas due to a rather high organic content, influence the compression. However, lateral movements of considerable magnitude have obvioulsy occurred.

In 1957 the Institute, in cooperation with the Swedish Roads Board, constructed a test field at Skå-Edeby on the site of a proposed airfield for the Swedish Board of Civil Aviation (cf. Osterman, 1959). The test field comprised four circular test areas, three of which were provided with sand drains. For the drainage work a steel tube with an inner diameter of 16 cm was used. Area I is 70 m wide and divided into three sections, with 0.9, 1.5 and 2.2 m drain spacings, respectively. Area II and III are 35 m wide and provided with sand drains, with 1.5 m spacings. Area IV is not drained. Area I, II and IV were loaded with 1.5 m gravel and area III with 2.2 m gravel, with a unit weight of 1.79 ton/m³ on an average. Area III was provided with a loading berm to ensure stability.

The soil consists of post-glacial clay to a depth of about 5 to 6 m. Underneath, there is glacial varved clay with thin layers of sand and silt near bedrock level. The depth to the bed rock varies from 9 to 15 m. The dry crust has a thickness of up to 1 m. The clay fraction ($< 2\ \mu$) varies between 60 and 70 per cent of the total dry wight, and the carbon content is about 0.4 per cent.

The lateral movements of the ground were measured immediately outside the test areas. The surface movements were measured on concrete blocks and the soil movements in plastic tubes, driven down to firm bottom. The tubes were intended to follow the movements of the soil. After some trouble with a measuring device which proved too stiff, the lateral movements were measured at certain points by means of a rod.

The lateral movement of the ground surface due to displacement by the driving of drains was 0.5–9 cm, on account of the drain spacings, and the movement due to the load was about 1.5–4.5 cm. For supplementary information about the measurements at this site reference can be made to Hansbo (1960).

Owing to irregularities of the soil conditions and in the size of the areas, the checking of the movements by measuring changes in water content gave no conclusive results.

On account of the shortcomings in the above measurements in the ground, the Institute has designed a new equipment (Kallstenius and Bergau, 1961) for measuring inclinations at different depths of a tube, installed in the soil.

Theoretical Considerations

When a stress system is applied to a saturated clay specimen, the volume changes in the particles and also in the water are, with few exceptions, disregarded. The deformations in connection with the loading of an undrained element are thus mainly regarded as shear strains. The volume formed by the loaded area multiplied by the initial settlement in this area should be equal to the volume involved in the lateral soil movement at the boundaries of the load. This means an oversimplification of the natural event, at which the volume of the soil in fact decreases.

The skeleton is now sagging under loading, and the load is mainly carried by pore water pressures. This means that, initially, the vertical effective pressures on the skeleton are only slightly affected.

The stresses exceed the creep limits and produce longterm deformations. The rate of the plastic deformations should logically be at a maximum at the beginning of the period of settlement.

During the sag of the skeleton, the pore water pressures are kept at high values, which involves a high pressure gradient and a high rate of water flow and shrinkage.

The consolidation is thus accompanied by a series of processes and it will last until the clay has become so compact that the skeleton can carry the load.

In the fundamental case in which lateral movement is prevented, the shearing and the shrinkage are linked together. This case may be quite acceptably simulated in the laboratory, provided that one can extract the test specimens with the very minimum of disturbance (cf. Swedish Committee on Piston Sampling, 1961).

In the more general case in which lateral movement may occur the shearing deformation obviously must dominate to begin with, but in the horizontal direction it is to some extent counterbalanced not only by shrinkage but also by an increase of the horizontal earth pressures of a passive nature. This case cannot be satisfactorily simulated in the laboratory.

However, at a certain time the rate of lateral movements due to shrinkage is as high as the rate of shear deformation, and the total movement will reach a maximum. Later on there will – to judge from the Skå-Edeby tests mentioned – be certain reverse movements during the time of shrinkage.

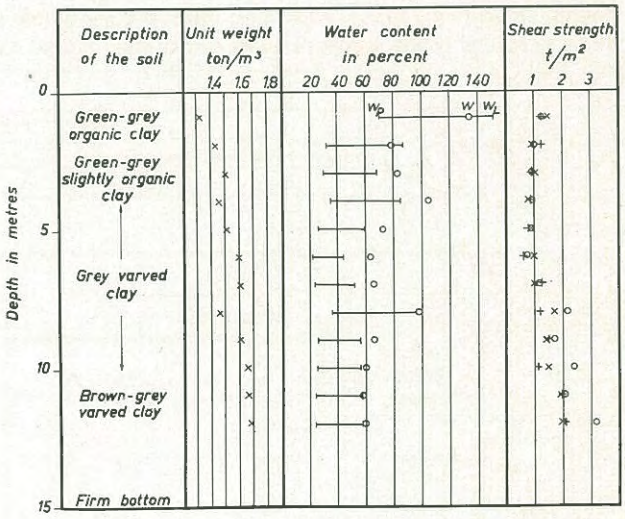
The relative influence of the lateral movements on the settlements is thus varying. Therefore, the relation between settlement and consolidation time will be affected, which seemingly has caused some trouble in the examination of field measurements.

If the soil below a loaded area had no connection with the surroundings, theoretically a discontinuity in the settlement at the boundary of the area should appear. In practice shearing stresses are transferred to the connecting surfaces of the soil at the boundary. The settlements here should, at a widespread load, approximate to one half of the settlement of the middle part of the loaded area. Other circumstances add to the stress strain complex at a local load.

Some results of measurements of settlements and the influence of lateral movement thereon are given in the following section of the paper.

Further Tests at Skå-Edeby

In connection with unloading the test area III at Skå-Edeby the surplus gravel was used for another loading test. These operations were sponsored by the Swedish Roads Board and the Swedish Council for Building Research. At the new test area the depth to the firm bottom is about 15 m. Below a thin dry crust the clay is rather soft. The shear strength is increasing somewhat with depth. Results of investigations into the soil properties are shown in fig. 2.



Symbols concerning shear strength

+ *Vane tests in the center line*
× *Vane tests 10 metres off center line*
o *Cone tests at center line*

Fig. 2 Loading tests at Skå-Edeby. Soil data

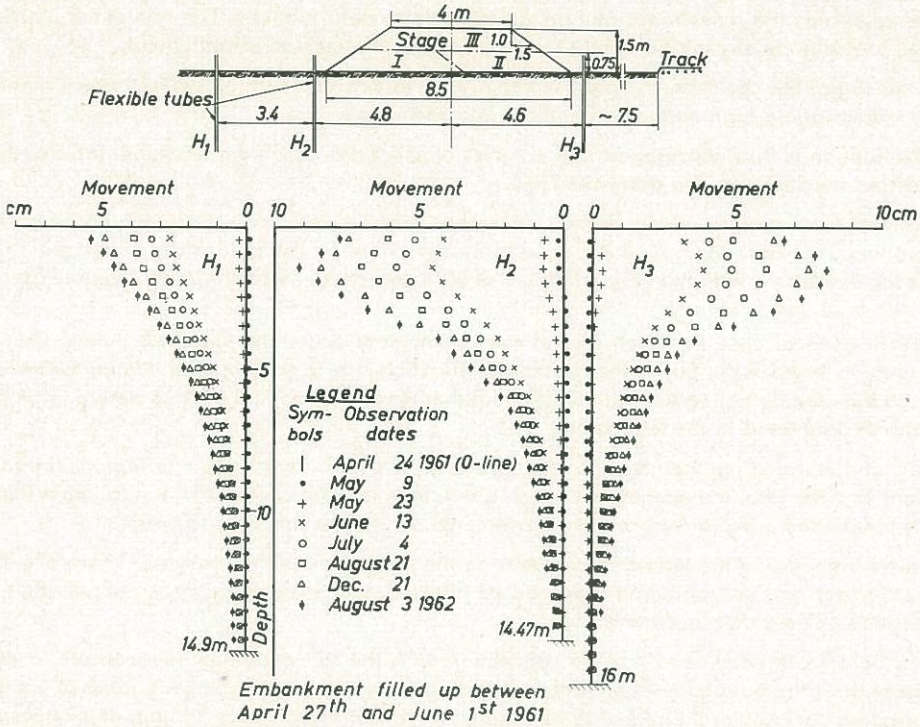


Fig. 3 Loading tests at Skå-Edeby. Horizontal movements in clay caused by a gravel embankment

The gravel was placed to form an embankment, 40 m long, 1.5 m high and with a crest breadth of 4 m. The slopes were inclined at a ratio of 1 vertical to 1.5 horizontal. In order to give time to make measurements at the filling stage, the embankment was erected in three stages (see fig. 3). The unit weight of the gravel became about 1.8 ton/m³.

Arrangements were made for measuring the movements in the soil and the pore water pressures below and outside the fill area. The measuring equipment was placed close to the middle of the embankment in order to get the conditions of a plane deformation.

The lateral movements in the soil were measured on both sides of the embankment in plastic tubes with the new equipment mentioned above. Although the tubes were rather flexible, some stiffness in the tubes and also in the measuring device may have slightly influenced the results. Special investigations of other possible measuring errors show that these are negligible in the actual tests. The arrangements of the tubes and the results of the measurements are shown in figs. 3 and 4.

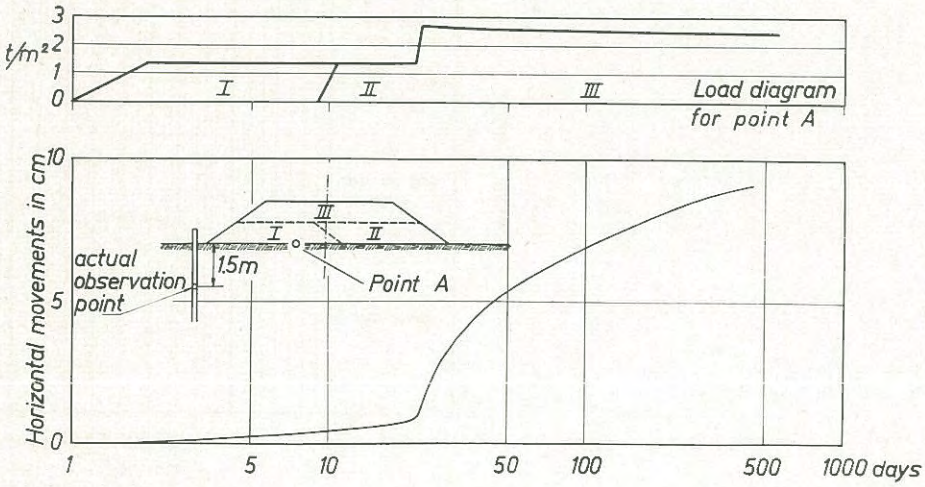
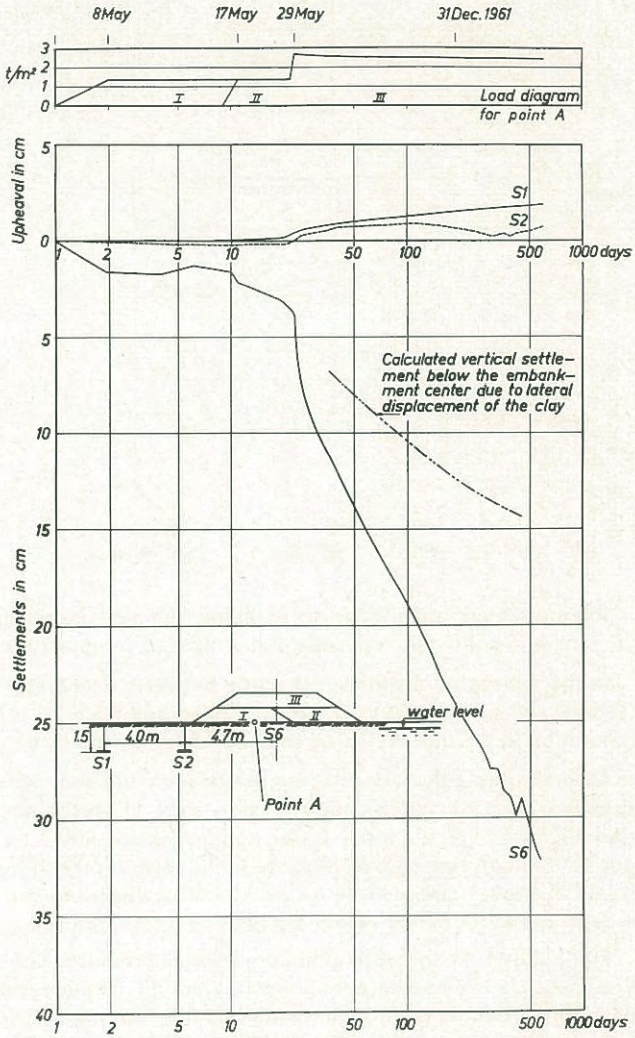


Fig. 4 Loading tests at Skå-Edeby. Horizontal movements in clay at a depth of 1.5 m just outside the gravel embankment

Fig. 5 Loading tests at Skå-Edeby. Vertical movements in clay caused by a gravel embankment



The settlements of the ground surface on the centre line of the fill were measured by levelling the top of steel rods welded to plates, which were placed on the soil below the fill.

Outside the loaded area, where the vertical movements were expected to be considerably less than below the fill, the measuring equipment consisted of steel tubes, welded to earth screws. A rod was forced through the tube to the firm bottom. The movement between rod and tube was measured by a dial gauge. The screws were placed at a depth of 1.5 m, in order to avoid climatic influence on the measurement results. The arrangement of some of the settlement meters and the test results are shown in fig. 5.

As mentioned above the pore pressures caused by the embankment were also studied. Pore water pressure meters, Type SGI II (Kallstenius and Wallgren, 1956), were placed below and outside the test area. Fig. 6 shows the location of some of the meters and the corresponding results of the observations.

From fig. 5 it appears that the settlement of the ground surface below the center of the embankment developed rather rapidly after load application. At the same time some upheaval appeared outside the slopes.

The clay volume which has yielded laterally is computed from the results of the observations in the plastic tubes. Supposing that the depression below the embankment in cross-section approximates to a parabolic shape, the influence of the horizontal displacement on the settlements of the clay has been calculated. The results are shown in fig. 5.

At the beginning the lateral movements in the clay were thus in this case the cause of a major part of the total settlement. With time this influence was, however, considerably decreased which can also be seen in fig. 4. In the figure the lateral deformation is plotted in relation to time for a point at a depth of 1.5 m below the ground surface and just outside the embankment.

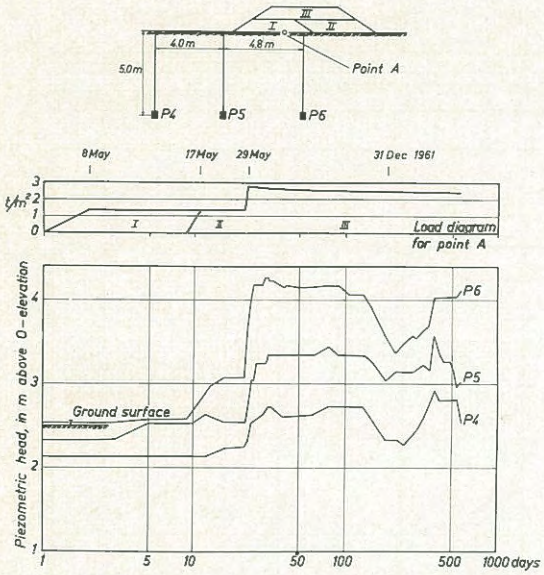


Fig 6 Loading tests at Skå-Edeby. Pore pressure measurements

Attempts have been made to correlate the settlements due to compression with the decrease of water content in the loaded soil body but it proved impossible mainly because of the heterogeneity of the soil.

In the figures no distinction is made between elastic and non-elastic deformation. In the unloaded area (similar soil conditions) the load was decreased from 3.1 ton/sqm to 1.9 ton/sqm, and the vertical recovery amounted to 7.5 mm including the aftereffect during three months.

According to calculations using the undrained shear strength of the clay (see fig. 2) the ground should sustain a load of about 4 ton/sqm at a state of plastic equilibrium. As the weight of the embankment is about 2.7 ton/sqm, the safety factor against failure would be about 1.5. However, the maximum shear stress due to the load has obviously exceeded a certain creep limit in some zones – locally approached even the shear strength – and caused an extra lateral displacement. The shape of the deflexion of the plastic tubes may to a certain extent reflect the shear stress in the soil.

Fig. 6 shows the increase of the pore water pressures at a depth of 5 m, when the embankment is applied. The increase is also perceptible outside the fill. The pressure decrease in the winter 1961–1962 may have been influenced by the frozen surface layer which might have distributed the gravel load over an increased area. Experience from the Skå-Edeby test field has also indicated that the recording of the pore pressures may have been affected by rapid climatic changes.

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Consolidation Tests on Clay Subjected to Freezing and Thawing

By J. G. Stuart, Queen's University, Belfast

Introduction

The problems associated with freezing of soil have been studied by many workers, especially the volume changes in fine-grained soils caused by winter freezing, or in the foundations of cold-storage plants etc. For the most part, the investigations have concentrated on the mechanism of pore-water migration from unfrozen regions in the soil which leads to formation of ice lenses and frost heave. The subsequent

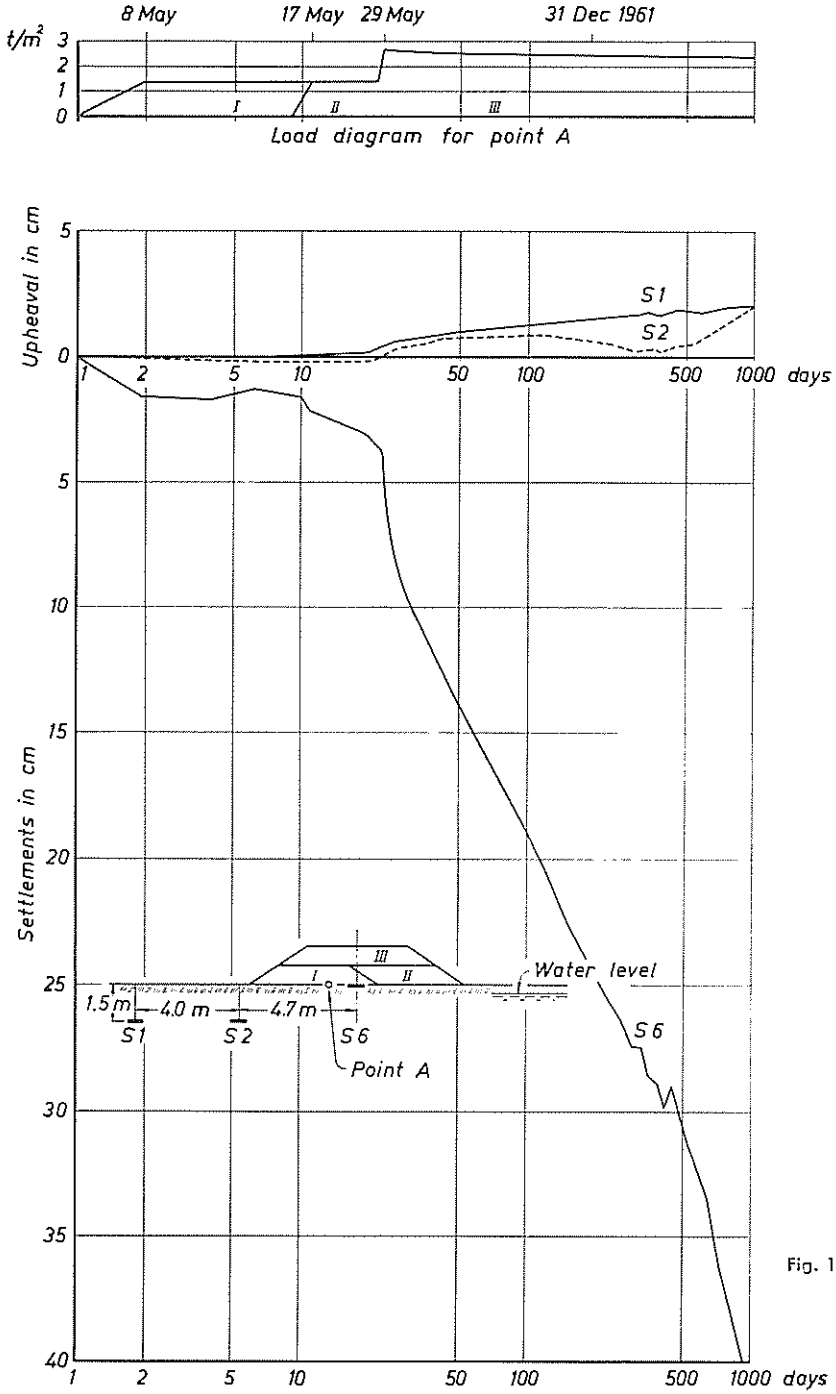

Fig. 1 Loading tests at Skå-Edeby. Vertical movements in clay caused by a gravel embankment

thawing leads to a greatly increased moisture content and loss of soil strength.

The present study arose after an examination of settlement records of a gravel embankment which had been constructed at Skå-Edeby as part of a field study being carried out there by the Swedish Geotechnical Institute. A description of the site conditions and of the bank is given by Osterman and Lindskog in their paper which is included in the present publication (cf. also Hansbo, 1960).

Observations on the bank have continued since then, and Fig. 1 shows the soil movements plotted up to the end of 1963. In the winter of 1961—1962, about 300 days after the test was started, there was a pause in the settlement of the bank and after that, the settlement curve, which up to that time had been smooth, become irregular and the rate of settlement increased. A change in trend of the curves relating to points outside the bank can also be seen. These observations cannot be attributed entirely to the after-effects of frost action; nevertheless it was thought that there could be changes in soil structure in the loaded upper layers following freezing, which might provide a partial explanation. The tests described in this Preliminary Report were therefore started to investigate the effect of freezing and thawing during consolidation tests on the clay.

Description of the Soil Samples

The bank at Skå-Edeby rests on a bed of soft clay some 15 m thick, the lower part of which is largely glacial varved clay and the top 3 m or so is a post-glacial organic clay with a dry crust of 1 m and the ground water level is about 0.5 m below the ground surface. The properties are given in detail by Osterman and Lindskog (1963). The samples for the present series of tests were obtained during the site investigation in 1962, and to begin with, samples from a depth of 8 m were used. Later when this material ran short, the series was continued using samples from both 7 m and 9 m below the ground surface.

Table I lists the samples and shows the results of classification tests which were carried out in September 1962 shortly after the site investigation; the results shown in brackets refer to tests carried out in September 1963, in connection with the present series of consolidation tests.

Table I Classification Tests

Sample tube No.	Depth m	Natural water content %	Liquid limit %	Plastic limit %	Fall-cone shear strength t/m²	Sensitivity	Unit weight γ gm/cm³
2105	7.0	72 (75)	—	—	0.770 (0.580)	(10)	1.57
2404	7.0	73	57	24	0.835	15	1.58
3174	7.0	68 (68)	—	—	1.00 (0.757)	(18)	1.60
2315	8.0	67 (69)	—	—	1.10	—	1.60
2737	8.0	70	74	46	1.13	18	1.57
2778	8.0	66 (68)	—	—	1.02	—	1.65
2030	9.0	78	—	—	1.25	—	1.63
2287	9.0	63	47	20	1.75	29	1.65
2598	9.0	61	—	—	1.67	—	1.63

Test Procedure and Results: Part 1, Freezing and Thawing Tests

The consolidation tests were carried out in the Institute's small, direct loading oedometers, a diagram of which is shown in Fig. 2. In this apparatus, the loading column is provided with a friction locking device so that the oedometer may be moved from one place to another if required. The diameter of the test specimen was the same as the sample tube (50 mm), so the soil was directly extruded into the consolidation apparatus with the minimum of disturbance. The initial thickness of the sample was 20 mm in all cases.

After the sample had been set up in the oedometer, a normal, standard consolidation test was carried out, up to a pre-determined pressure. When the sample had come to equilibrium, the loading stem was locked while the apparatus was moved to the deep-freeze box, where it was levelled and immediately

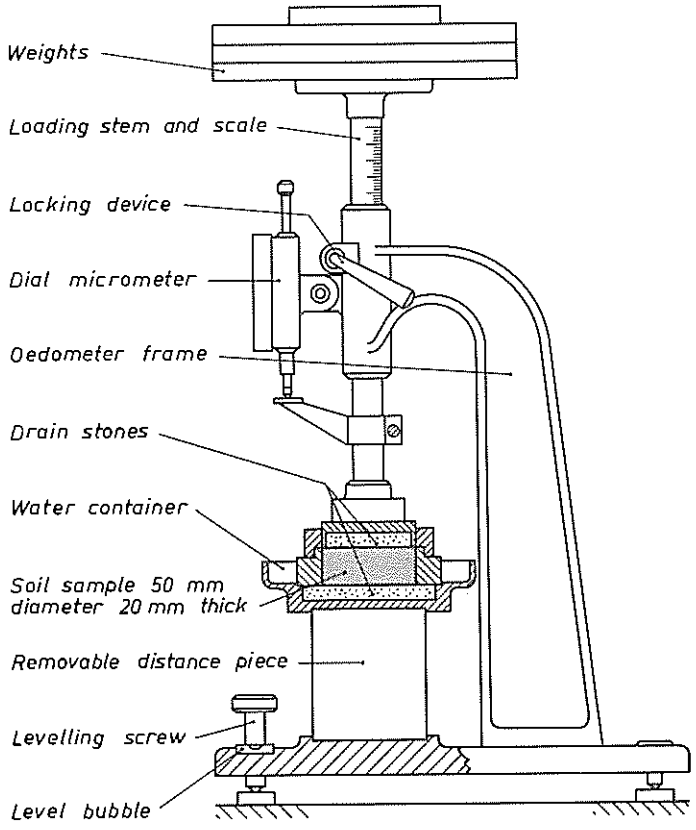

Fig. 2 Diagram of apparatus. Direct loading oedometer

unlocked so that the test continued under the same loading. Readings of the dial gauge continued until no further movements were observed (usually over a period of seven hours). The next day the apparatus was removed from the freeze box and allowed to thaw out in the laboratory while further readings were taken.

The freezing cycle temperatures were usually from room temperature, in the region of +20° C, to the freeze box which operated at —18° C, and then back to room temperature. In one case the test started and finished at +8° C but the same freezing temperature was used in all the tests. In some cases the freezing and thawing cycle was repeated over the same temperature range.

After the test was completed, the sample was removed and tested for moisture content, strength and sensitivity. Table II which follows, summarizes the tests made.

Typical plots from two of the tests (samples 2315 and 2105) are shown in Fig. 3. The consolidation test before freezing follows the usual pattern and requires no special comment here. From the time when the apparatus and sample started to cool, a slight further apparent settlement was observed. Calculations show however, that this small movement was largely due to changes in the dimensions of the oedo-

Table II Classification Tests after Freezing Cycles

Sample tube No.	Maximum pressure before and during freezing t/m²	Number of freezing cycles	Sample condition after test		
			Moisture content %	Fall-cone strength t/m²	Sensitivity
2105	2.01	1	41	2.79	5.5
2105	32.48	1	36	3.74	13.0
2315	6.01	2*	36	3.38	1.9
2778	6.01	1	34	4.32	5.1
2778	6.01	2	32	4.19	4.7
2598	15.98	1	33	3.46	3.9

* Test started and finished at +8° C

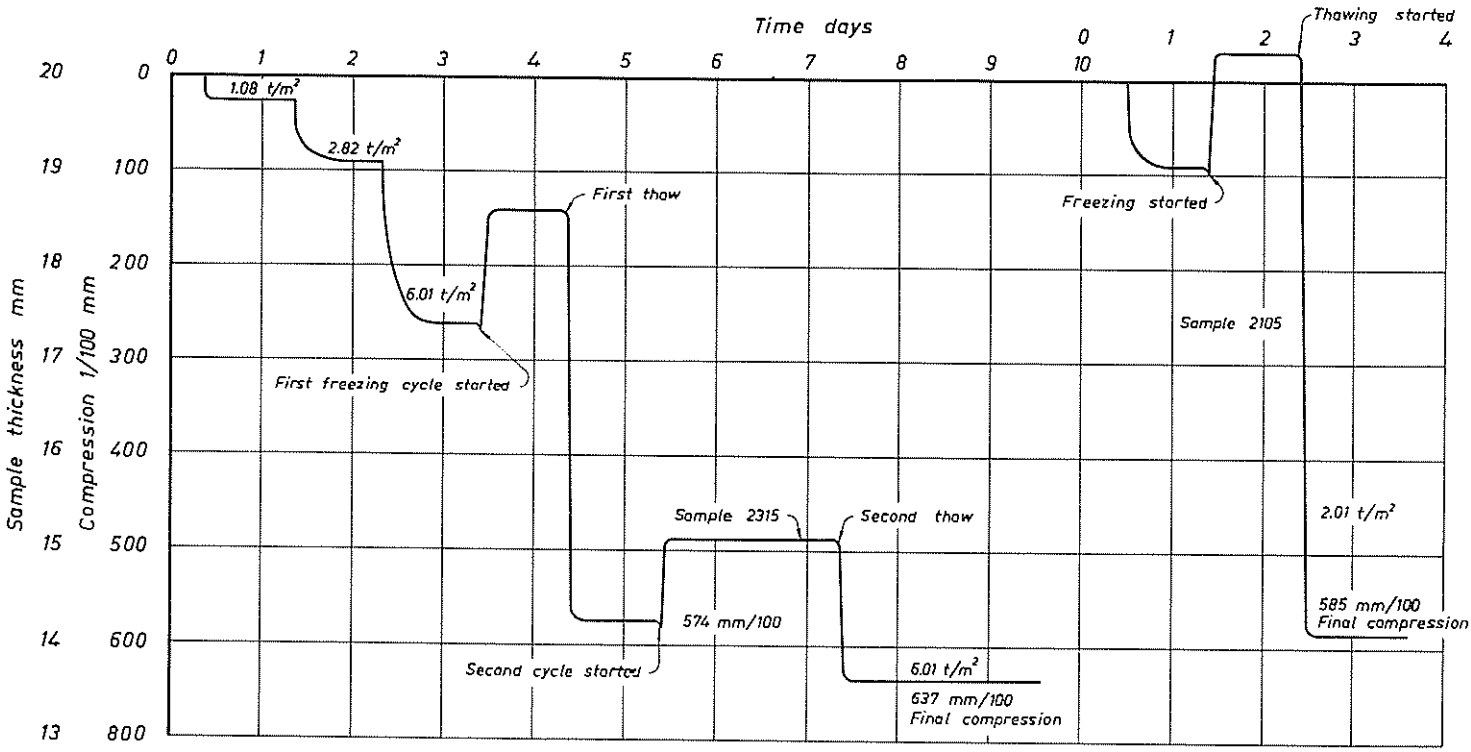

Fig. 3 Typical time-compression curves for consolidation tests on samples from Skå-Edeby (subject to freezing and thawing cycles)

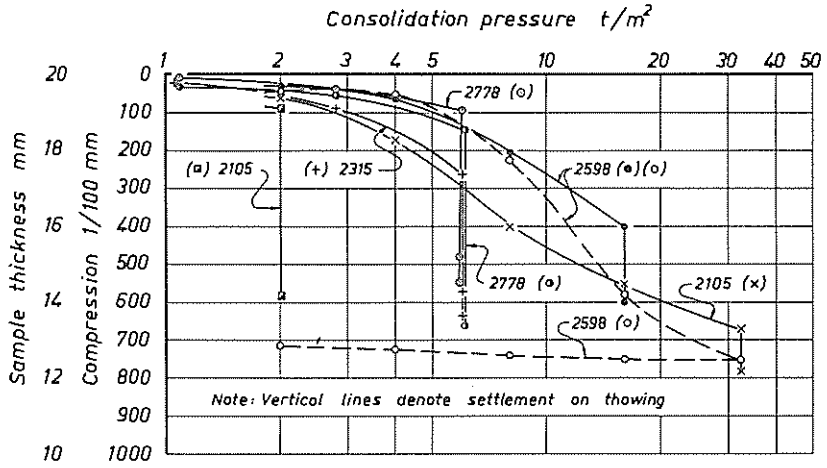

Fig. 4 Load-compression curves for Skå-Edeby samples (freezing and thawing tests)

meter as it cooled. After about three hours the sample started to freeze, and an expansion, usually of the order of a millimeter was observed; the greater part of the movement took place over an hour and a half to two hours. Although all the samples were frozen under 'open circuit' conditions, that is to say, were able to take up more water during freezing, the amount of expansion corresponds in all cases to the increase in volume to be expected from the freezing of the pore-water alone. This shows that the rate of freezing in conjunction with the low permeability prevented true frost heave taking place.

The freezing process usually took the greater part of a day, and next morning, after checking that no further volume changes had occurred, the apparatus was replaced in the laboratory to thaw out under the same load. During thawing further rapid consolidation of the sample took place, and was usually so quick that it masked the dimensional adjustment of the apparatus to the higher temperature. In all cases the final reduction of sample thickness was of the order of 5 mm irrespective of the maximum consolidation pressure before freezing. In the two cases where the freezing cycle was repeated, the further settlement recorded on thawing was small by comparison and was about 0.3 mm. The load-compression curves for the freezing tests are given in Fig. 4.

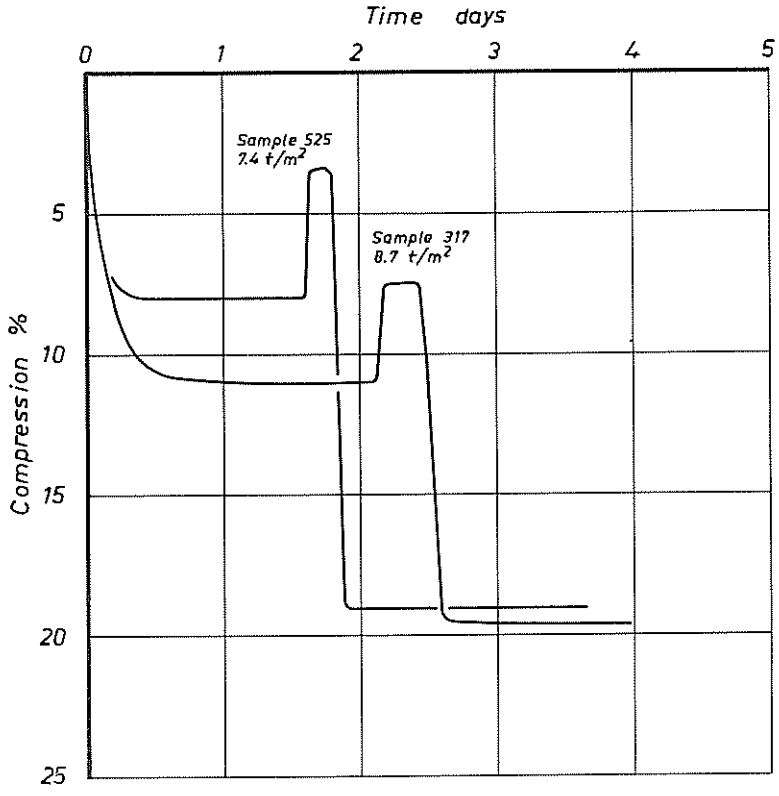

Fig. 5 Compression tests with freezing and thawing on clay samples from Centralplan, Stockholm (K 4470)

During this series of tests, it was remembered that a somewhat similar testing programme had been carried out in 1955 (SGI Arkiv No. K 4470) on samples of clay soil from a site in Stockholm in connection with a tunnel for the Underground Railway at Centralplan. It had been noticed that after artificial freezing of the ground, surface settlements after thawing were greater than the heave associated with freezing. Similar observations have also been reported from Arlanda Airport. The Stockholm results are shown in Fig. 5. The rate of freezing and the lowest temperature reached were not as great as in the Skå-Edeby tests, but precisely the same pattern may be seen.

Test Results: Part 2, Samples Disturbed during Consolidation

The tests described above, suggested that the clay, during the freezing cycle underwent a change corresponding either to a disturbance of structure or to a compression due to extraction of water to form thin ice layers. To investigate these possibilities two further groups of experiments were carried out. In the first the sample was consolidated as before, and then stirred in-situ after which the test was continued under the same pressure. In the second test a sample was compressed using a higher load than usual followed by a gradual reduction of load to give an expansion curve.

For the 'disturbed' tests the procedure was as follows: after the standard consolidation test up to the required pressure, the apparatus was quickly dismantled and the sample exposed at the upper surface by removing the top porous plate. It was then stirred carefully in an attempt to remould without introducing air. Following this, the apparatus was re-assembled and the test continued under the same pressure which had been applied immediately before dismantling. A careful check on the volume increase following disturbance was made by reading the scale on the loading stem before and after stirring. A typical time-compression curve for one of the samples is given in Fig. 6. This diagram may be compared with Fig. 2 where the results of a freezing test on clay from the same sample tube, and loaded to the

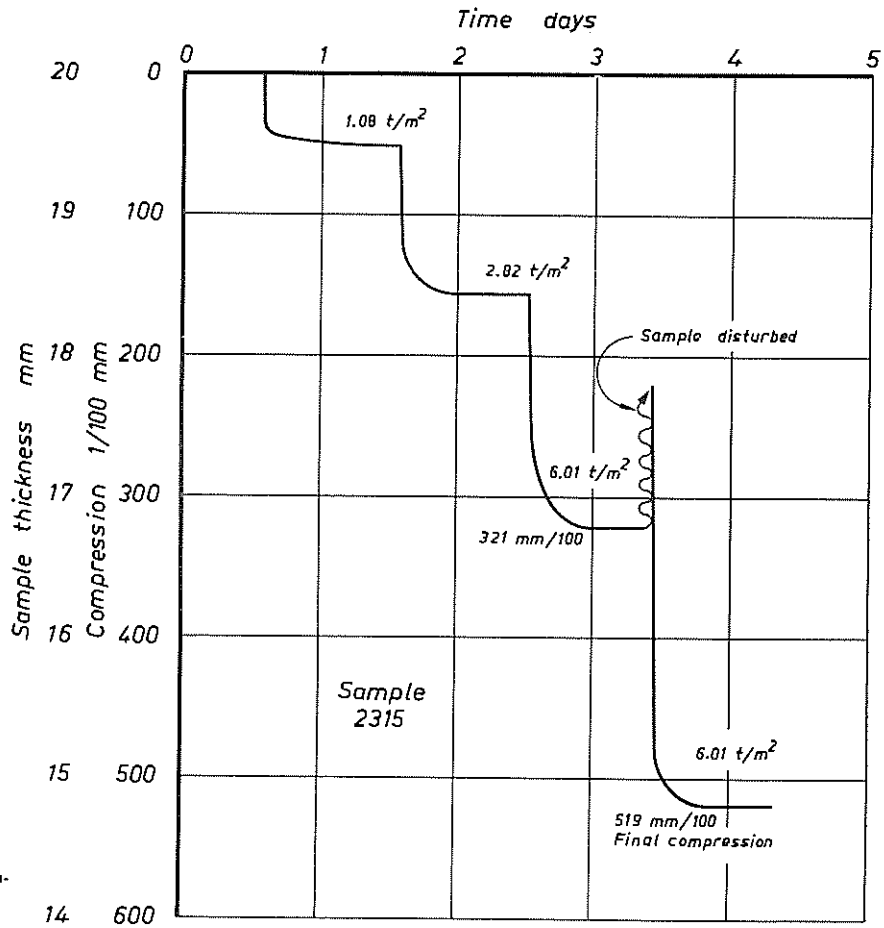

Fig. 6 Typical time-compression curve for consolidation tests on samples from Skå-Edeby (unloaded, stirred and reloaded)

some consolidation pressure is illustrated. The load-compression curves for these tests are shown in Fig. 7, and a summary of these tests together with the results of classification tests made before and after consolidation is given in Table III.

The standard consolidation test, the results of which are included in Fig. 4, was carried up to a sufficiently high load so as to give approximately the same total compression of the clay as the freezing tests, viz. about 5 mm. In fact the change in thickness was greater than intended and came to 7.57 mm with a pressure of 32 t/m². The intention was to provide an unloading curve to compare with the final compression after freezing and thawing of the first test series.

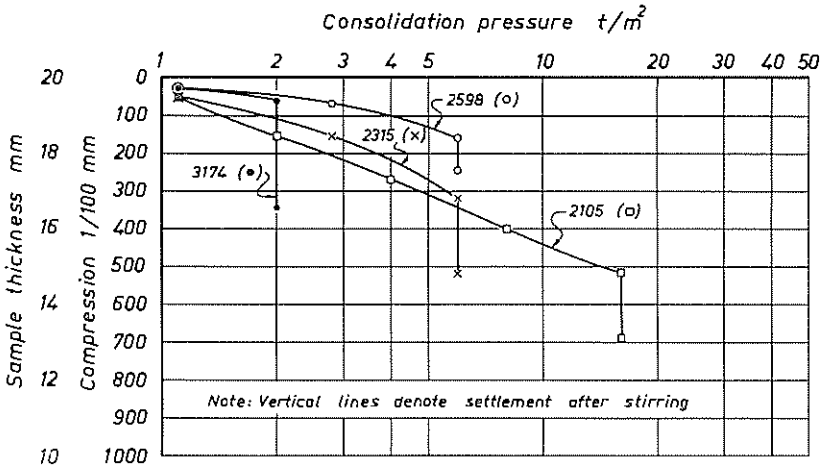

Fig. 7 Load-compression curves for Skå-Edeby samples (remoulded by stirring)

Discussion of Results

Results of freezing tests carried out in 1935 by Casagrande showed that the moisture content of clay layers between ice layers in soil subject to frost heave, was significantly reduced as a result of water migration under the influence of 'ice crystallisation pressures'. Work on this effect has been carried out recently by Williams (1963) and Fredén (1964) which shows that during freezing of the soil the pore water may develop negative pore pressures which depend on the temperature. Williams suggests that the consolidation which results, together with the discontinuities left by ice lenses give rise to special structural features in soils subjected to freeze-thaw cycles. In the tests described above, no true frost heave occurred since no water came into the sample to form ice lenses, but it would appear that some water did actually separate from the clay, resulting in a pre-consolidation effect when the clay thawed. An alter-

Table III Results of Consolidation Tests — Samples Disturbed during Test

Sample tube No.	Maximum pressure t/m²	Compression before stirring mm	Increase of sample thickness after stirring mm	Final total compression mm	Moisture content %		Fall-cone strength t/m²		Sensitivity	
					before test	after test	before test	after test	before test	after test
2105	15.98	5.17	0.80	6.95	75	34	0.58	3.84	10	1.7
3174	2.01	0.56	0.60	3.43	68	56	0.757	0.90	18	5.0
2315	6.01	3.21	1.00	5.19	69	42	(1.10)	2.13	—	3.9
2598	6.01	1.62	0.40	2.45	(61)	38	(1.67)	2.13	—	4.6

Notes: Figures in brackets refer to tests made in September 1962
Original thickness of samples 20 mm

native explanation for the large amount of compression on thawing is that the freezing cycle altered the clay structure so there was a change from 'undisturbed' to 'disturbed' consolidation characteristics, and there is some evidence for this in the reduction of the sensitivity of the clay after freezing. The rate of consolidation on thawing was most marked, and can only be explained in terms of either a very large increase in the permeability or the water having already separated from the clay skeleton during freezing.

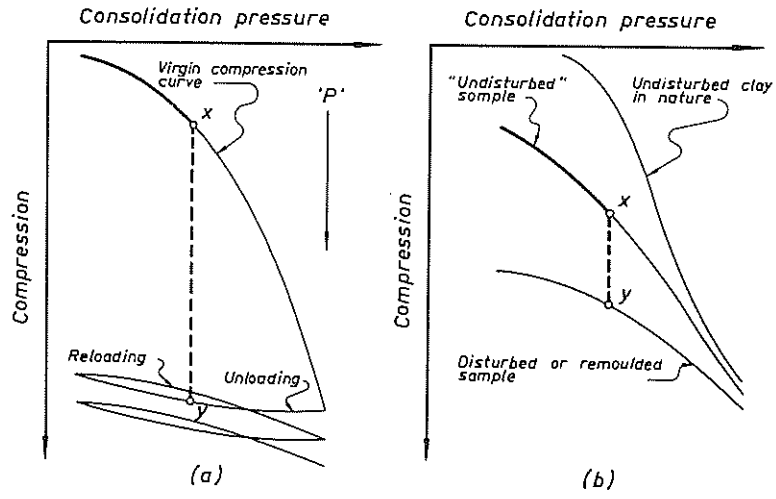

Fig. 8 Sketches illustrating hypotheses of freezing effects

In Fig. 8 two hypothetical sets of consolidation curves have been sketched. Fig. 8 (a) shows a curve such as might be obtained by consolidating a clay soil to a given pressure 'P' followed by a number of unloading and loading cycles. The second set of curves shown in Fig. 8 (b) illustrates the differences in the consolidation curves of a clay resulting from different degrees of disturbance. Suppose a sample of clay is consolidated as shown by the thick line to the point 'x', and is then frozen. On thawing further consolidation takes place under the same pressure and the sample condition moves to point 'y'.

The 'pre-consolidation' hypothesis means that if 'P', Fig. 8 (a), is equivalent to the 'ice crystallisation pressure' (which according to Casagrande might depend on the temperature and the rate of freezing) then point 'y' will lie on the unloading curve. Furthermore, samples frozen at various pressures 'x' will consolidate on thawing so that the points 'y' will lie on the unloading curve. Since the unloading curves for a clay are more or less parallel, then even if pressure 'P' is not equivalent to the 'ice crystallisation pressure', the unloading curve should nevertheless lie more or less parallel with a line through the points 'y'.

The 'disturbance hypothesis' means that after freezing and thawing, the points 'y' should lie along the consolidation curve for the disturbed condition in Fig. 8 (b). A comparison of Figs. 4 and 7 seems to indicate that the frozen samples come to equilibrium on thawing, along a line roughly parallel with the unloading curve from the test on sample 2598 which was unloaded from a pressure of 32 t/m², rather than along the line taken up by the stirred samples.

It might be argued that the scatter is too large to draw any final conclusions, some indication of the original variation between samples being provided by the liquid and plastic limits given in Table I. A rigorous investigation of the problem in the laboratory would entail the preparation of a quantity of sensitive clay, and that has been outside the scope of the present work.

The measurements of the bank movements and the frost penetration at Skå-Edeby have been too incomplete to permit a reliable comparison between the effects of freezing and thawing in the field and those described above. The frost penetration is seldom more than one meter, and the soil directly under the bank would have been insulated so that the depth of frost penetration was less, and only a thin layer of clay would have been subjected to freezing and thawing, so that only a small part of the changes in the settlement pattern can be accounted for. The problem at Skå-Edeby is also complicated by the horizontal soil movements which have been observed. These might be due not only to consolidation but also in part to plastic shear movements or creep in the soft ground, and the investigation of these problems is outside the scope of the present work.

Conclusions

The results of the tests show that there can be a significant reduction of the moisture content of a clay when it is frozen which can give large settlements on thawing, provided the loading conditions remain constant. The rapid consolidation may either be due to a partial remoulding effect or disturbance of the clay structure, or more probably to a separation of the water from the clay skeleton during freezing so that the clay is effectively pre-consolidated. On thawing, provided the water (whether from the original clay structure, or, it is suggested from ice lenses arising from frost heave) can escape, the clay will rapidly consolidate to a volume corresponding to the 'ice crystallisation pressure' which in turn appears to be a function of the rate of freezing and the temperature.

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