Consolidation of clay by band-shaped prefabricated drains

Discussion on "Consolidation of clay by band-shaped prefabricated drains" by Sven Hansbo, published in this journal, July 1979, Vol. 12 No. 5, pp 16-25.

by BENGT H. FELLENIUS*, PEng., DrTech, MEIC, MASCE

IN PRACTICAL design work, there is little reason for using any more complicated design formulae than the one originally proposed by Kjellman (1948).

$$t = \frac{D^2}{8\, c_h} \cdot \left(\ln \frac{D}{d} - 0.75\right) \cdot \ln \frac{1}{1-\bar{U}}$$

where t = time,
c_h = horizontal coefficient of consolidation,
D = zone of influence of a drain,
d = equivalent diameter of a drain and
$\bar{U}$ = average degree of consolidation

For a band-shaped drain installed at normally applied spacings, the difference in time calculated from the above equation, as opposed to the complete equation No. *1a* given in the Paper by Hansbo, is a mere 2%, which is negligible.

The degree of deviation from truly square or triangular patterns of installation will have a much more appreciable effect on the results of the drain installation, i.e. the agreement between predicted and actual behaviour. For instance, calculating the consolidation time for a square pattern, as opposed to a triangular, results in an increase of consolidation time by about 20%. On the other hand, for equal consolidation times, the calculated number of drains per unit area is the same, whether assumed as placed in a square or a triangular pattern.

Fig. 1 shows a nomogrammetrical representation of the design formula for a drain with an equivalent diameter of 15cm installed at a spacing of 140cm in a soil that has a horizontal coefficient of consolidation of 4 x 10^{-4} cm²/s. For these data, the nomogram indicates that 50% average consolidation is achieved in 100 days. However, the time is directly dependent of the c_h value of the soil, which is very difficult to determine precisely. Errors, or rather imprecisions, of one order of magnitude, or more, are not unusual. Therefore, an error of 2, i.e. a real c_h value of 8 x 10^{-4} cm²/s for a case where c_h = 4 x 10^{-4} cm²/s was assumed, is considered to be quite acceptable. It does however, have an equally large influence on the calculated times, putting the usefulness of refined theories and formulae in doubt.

Not only is the value of c_h uncertain. but so is also the value of $\bar{U}$. To find, for instance, that the truly predicted settlement, i.e. calculated in advance of the construction, is within about 30% of the actual settlement must be considered a good agreement. Suppose now that for two cases where settlement values of 100cm were predicted, the actual total ($\bar{U}$ = 100%) consolidation settlement was 100 x 0.7 = 70cm in one case and 100÷0.7 = 140cm in another. (That is, an observed settlement of, say, 50cm would be interpreted to represent U = 0.50, whereas in reality it is 50/70 = 0.71, and 50/140 = 0.35, respectively). Suppose also that for both cases the actual c_h value was 8 x 10^{-4} cm²/s instead of the 4 x 10^{-4} cm²/s assumed in the design. Such variations between the real and the assumed values are significant, but far from uncommon. However, as illustrated in Fig. 2, the designer who monitors the construction to verify his design will in both the supposed "real" cases find a confirmation of his design. The deviation shown in the nomogram would certainly be considered quite minor. In other words, a prediction based on a combination of values, where each is wrong by factors of 0.5 or 2.0, would not be noticed in the results!

When the monitoring is limited to a simple study of the settlements with time, and compared with predicted settlements, as shown in Fig. 3, an acceptable agreement can again be found between the predicted and actual settlements. When a settlement of 60cm has been reached almost within the week of the predicted time, the subsequent infinitely long time required to reach the 70cm value would probably not worry the designer.

In the unusual event that the actual settlements are greater than the predicted, this will not be noticed until after almost a year after the start of the project. Then, the large continued settlements would, probably, be "explained" by claiming the presence of a large secondary compression, and the data be used to confirm the analysis, anyway.

The example illustrated in Figs. 2 & 3 represents the insensitivity of the theoretical design. A general mathematical relation for the error in c_h needed to compensate for an error in $\bar{U}$, or vice versa, to arrive at the same calculated time is given in Fig. 4. This diagram shows that when the degree of consolidation is smaller than about 0.50, or when the assumed $\bar{U}$ is smaller than the real, the compensating error in the coefficient of consolidation is small. On the other hand, when the degree of consolidation is near the value of one, and the error of $\bar{U}$ is one of over estimation ($\beta\bar{U} < 1.0$), a large compensating variation of c_h is required.

It is obvious from the foregoing that reliable conclusions cannot be drawn from field measurements until the consolidation is nearly finished. Until this is the case, agreements between predicted and back-calculated behaviour may only be apparent due to compensating errors.

The above discussions have made use of settlement observations only. In prac-

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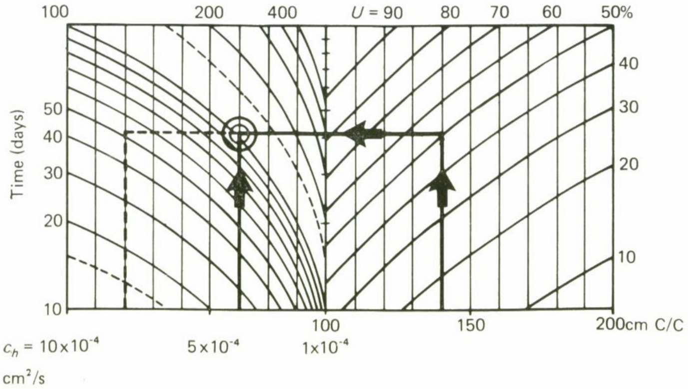


Fig. 1. Nomogram for the spacing (C/C), the average degree of consolidation (U), and the consolidation time (days) for a 15cm drain

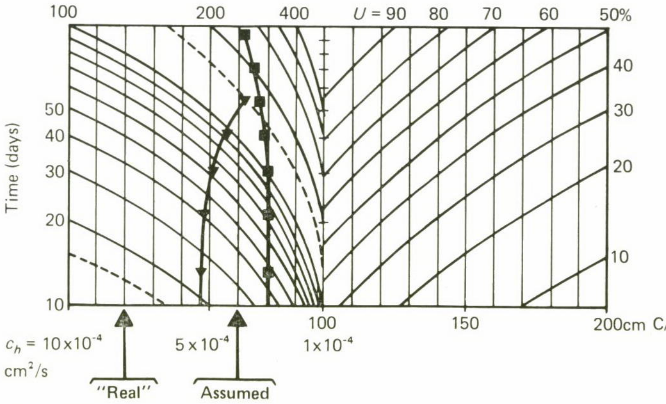
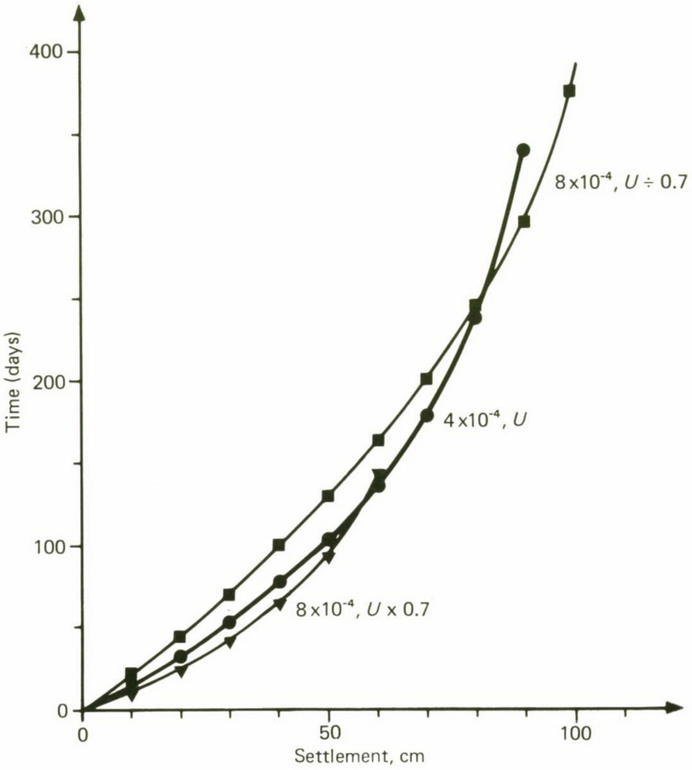


Fig. 2. The c_h value evaluated from "monitored" settlements, when using a 15cm drain at 140cm spacing, and when assumed c_h value is 4 x 10^{-4}cm²/s, but "real" is 8 x 10^{-4}cm²/s, and when assumed U is 100% at 100cm settlement, but "real" is U x 0.7 (▼) or U ÷ 0.7 (■)



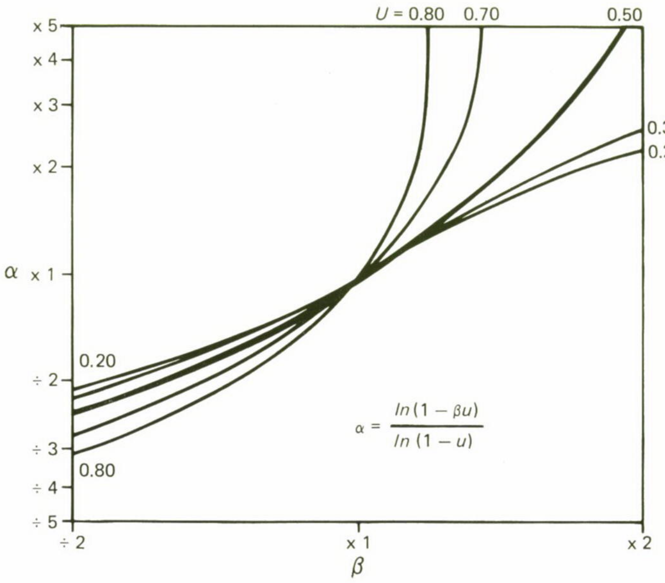


Fig. 3 (left). Settlement with time
● *Assumed.* $c_h = 4 \times 10^{-4} cm^2/s$, $U = 100\% = 100cm$
▼ *"Real".* $c_h = 8 \times 10^{-4} cm^2/s$, $U' = U \times 0.7$
■ *"Real".* $c_h = 8 \times 10^{-4} cm^2/s$, $U'' = U \div 0.7$

Fig. 4 (right). The necessary compensation in c_h (α) for an error in U (β)

tice, the settlement observations are usually supplemented by pore pressure observations. Apart from the fact that pore pressure observations can easily be erroneous, the two systems of determining the degree of consolidation do not agree, as is also mentioned by Hansbo. This provides additional difficulties in verifying the theoretical design from monitored data.

Note that the comparisons made in Figs. 2, 3, and 4 assume, optimistically, that the theories give a totally correct representation of the real time-settlement behaviour, and that the only errors involved are those caused by the wrongly assumed value of $\bar{U}$ in both absolute and relative terms, and that of the c_h value.

However, apart from building on the assumption that the Terzaghi consolidation theory (with its own inherent assumptions and simplifications) is valid, the design formula relies on an equivalent drain diameter of the band-shaped drain, which is rather cursorily defined.

Usually, the equivalent drain diameter of a band-shaped drain is simply determined as the diameter of a circle having a circumference equal to the perimeter (surface) of the band-shaped drain. However, this definition is not indisputable. In practice, the equivalent diameter has been determined from a variety of other assumptions — for instance, as the average of the length and the width of the cross-section, as the diameter of a circle having the same circumferential surface as the drain, as the diameter of a circle having the same circumferencial surface as the net (open or free) surface of the drain, or, finally, as the diameter of a sand drain whose net surface is equal to the net surface of the band-shaped drain.

The writer prefers the last of the above definitions. However, theoretical analysis by means of flow nets indicate that the net surface approach has less effect than previously assumed, although, as yet, there are no conclusive practical studies which could determine the best definition one way or another.

Hansbo emphasises in five valid points the importance of having a sufficiently good filter. However, although point No. 1 — that the filter permeability should not be considerably less than that of the soil in which the drains are installed — is correct *per se*, the title of the Paper "Consolidation of clay . . ." implies that this filter permeability can be in the range of that of clay, i.e. about 1×10^{-7} cm/s. However, clay soils usually contain bands, seams, lenses, or even continuous layers of much more pervious soils, e.g. silt and sand, with a permeability which is about 100 to 1 000 times greater than of clay. Then, the drainage in the clay is toward the more pervious zones, which in turn are dewatered by the drains. To be reasonably efficient, the filter must, therefore, have a permeability which is at least equal to that of silt or silty soils, i.e. about 1×10^{-4} cm/s.

All analysis of drain performance assumes that the water collected by the drain along its length in the soil can be discharged at the "ground water table", i.e. the elevation of zero pressure (= atmospheric pressure). The discharge elevation is often below the top of the drain, which means that the water collected must be discharged through the filter and not through the open end of the drain. The assumption postulates that the water collected under considerable pressure through the filter along a length of, say, 10 metres and more can be discharged at low pressure along a length of only a few centimetres. If the filter is insufficiently pervious, the water will rise in the drain inside the filter until a sufficient height is reached to achieve a discharge that balances the inflow. In actual projects water has been observed to rise as high as 2m above the elevation of zero pressure, when using drains with inadequate filter permeability. The back pressure so created eliminates much of the drainage effect and slows down the consolidation rate considerably.

Point No. 4 states that the filter must be strong enough not to break during installation, which is an important requirement. Equally important is that the drains can withstand the unavoidable abuse on a construction site. The drain rolls are exposed to rain, hot sunshine, as well as freezing weather. They are dragged on a truck floor and on the ground, stepped on, etc. Unless the filter is strong and wet-resistant enough, it will not only break, it will tear and be holed. At best, this will result in a large and costly rejected volume of rolls, and at worse, if undetected, result in the installation of damaged drains which could jeopardise the entire project.

Naturally, as pointed out by Hansbo, when using a drain with a thick filter, or a drain consisting only of a filter, the first requirement for the drain to be generally reliable is that the permeability of the filter is not less than that of the soil, i.e. equal to the permeability of silt or silty soils. It must be considered that such drains are very susceptible to the outside soil pressure and, thus, to installation depth and overburden pressure. When the soil pressure increases, the drain compresses, and the permeability decreases considerably with the compression. Tests on filter drains used as vertical drains, which were subjected to an outside soil pressure increasing from $0.1 kg/cm^2$ to $3.0 kg/cm^2$, indicated that the permeability of the filter reduced by an order of magnitude as a result of the compression.

The Paper by Hansbo gives credit to Mr. W. Kjellman, the inventor of the Kjellman cardboard-wick drain. However, it does not give due credit to Mr. O. Wager, who worked with Mr. Kjellman, and in about 1970 used Kjellman's principles to invent the Geodrain, which was a considerable improvement of the cardboard wick, and who later further developed his drain to the Alidrain.

It is hoped that this somewhat negative discussion will not leave the reader with the impression that the theories are of little value. The message is instead that it is the theoretical analysis with insufficient or inadequate field data which has limited value, and that the neglect lies in the field, not in the theories.

Further comment

by J. C. BRODEUR*, PEng, MEIC., MASCE

WITH REGARD TO the design views and recommendations presented by Professor Hansbo, this writer would like to add some comments from a background as a manufacturer of vertical drains, and a contractor for their installation.

Only rarely will all the details about the soil conditions at the site, which are pertinent for the installation of vertical drains, be known. A site is often investigated by a minimum of boreholes spaced 25 to 50m, or more, apart. These spacings should be compared with usual spacings between the drains, of about 1.5 to 3m. Therefore, even at a site where the borings and geological indications suggest uniform soft clay, there can exist pockets, seams, or layers of coarser soil. These materials can, firstly, cause considerable installation difficulties. Secondly and more importantly, the drainage of the clay in this case will be almost entirely from the coarser materials — first draining the clay and then discharging the water to the portions of the drains which are in the zones of the coarser materials — instead of, as assumed in the design, the water going radially and uniformly from the clay to the drains.

It should be recognised that bands and seams of silt or sand commonly occur in sedimentary clays, and that these also, when quite thin — a few mm only — determine the development of the consolidation of a site where vertical drains are installed. Consequently, the drains must have both a filter of sufficient permeability to accept the water, and a free surface area large enough to, locally, accommodate a substantial flow of water.

The writer, therefore, takes issue with the statement that the filter needs only have a permeability equal to that of the surrounding soil, assuming that the statement means equal to that of clay. The filter permeability must be at least equal to that of silt, i.e. more than 100 times greater than that of clay, or about 1×10^{-4} cm/s.

The filter characteristics are most important factors of a functionable drain. Professor Hansbo reviewed the aspect from the point of view of design only. However, based on actual experience, the practical aspects of handling, transporting, and resistance to weather influences are equally important factors and they must also be considered. The writer's actual experiences with filter cracking, tearing and breaking during rough handling and installation have proved extremely costly. Any installation of undetected defective drains, should they escape the site supervision personnel, will result in a reduced drainage effectiveness in the area involved. It is therefore imperative that these weaknesses in a filter be minimised or eliminated.

The first filters the writer used were made of paper. However, the filter requirements for permeability, wet strength, and durability are mutually defeating. A sufficiently pervious paper filter has too low a wet strength, whereas a sufficiently strong paper turns out to be practically impervious. But the durability of paper is such that even in ordinary soils, despite impregnation, it cannot be guaranteed to last for an extended period of time. Although filters made from synthetic materials are more expensive than those made from paper, the writer found that the paper filter had to be abandoned.

With a world-wide acceptance of prefabricated drains, a drain designed for a local clay soil condition is limited in function and effectiveness in sedimentary clays with silt or sand seams which are commonly encountered in many parts of the world. Mr. Oleg Wager, formerly of the Swedish Geotechnical Institute, soon realised the limitations of the early type of drains. Based upon the practical experiences referred to in this report, he resolved many of the earlier problems, using the concepts of larger free surface area and greater free volume, which more readily and reliably satisfy the conditions encountered in many areas of the world. In three such widely separated areas in Hawaii, Utah and Mississippi, in the United States, where spacings and depths varied, and exceed 6m and 40m, respectively, these new concepts were significant.

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Reply

by SVEN HANSBO

THERE IS NO difference of opinion between Dr. Fellenius and myself in respect of the practical difficulties involved in the design of a vertical drain installation and I fully agree that no theory, however refined, can compensate for grave mistakes in the choice of input parameters. The latter is self-evident and could be said about any kind of theory used for prediction of real behaviour, not least about, for example, the finite element analysis that is so popular nowadays. Nevertheless nobody would dream of denying, for these reasons, the opportunities for increasing our understanding of the mechanical behaviour of soil and other bodies that has been created by the finite element method. The same statement is of course also true with respect to the development of a theory which accounts for how the parameters involved in a vertical drain analysis influence the final theoretical result.

Fellenius presents an interesting example in which he shows that compensating errors might lead to wrong conclusions regarding prediction vs. performance. One could easily add to his example several other possible combinations of compensating errors. Since "simple settlement measurements" represent the easiest way of monitoring the consolidation process it is very important (as I pointed out in my Paper) that the settlement analysis be carried out with utmost care. That it is possible in practice to make a good prediction of the final primary consolidation settlement — with an error of not more than about 5-20% of the real value (cf. Holtz & Broms, 1972) — is beyond doubt. This would correspond to an error of not more than 2-10% at $U = 50\%$ which has a very small influence on the back-analysis.

It is true, as Fellenius states, that eqn. *1a* in my Paper is unnecessarily accurate if used for the design of a prefabricated drain installation of the conventional type of band-shaped drain. This fact is also mentioned in my Paper and it has therefore been given in simplified form, eqn. 4, which is the same equation as that presented by Fellenius. The reason for giving the more accurate value is that eqn. *1a* can also be used for other drain types with considerably larger drain diameters. Pocket calculators have also more or less eliminated the need for simplification of equations of the type shown since they are so easily programmable. But what Fellenius seems to have misunderstood is the importance for the process of consolidation of well resistance and smear. An example will illustrate this. Assume that we are dealing with a clay layer with a coefficient of consolidation of $c_h = 0.3\text{m}^2/\text{yr}$ and a permeability of $k_c = 0.03\text{m/yr}$ and that, in this clay, drains with a diameter of 0.066m have been installed at 1m spacing (triangular pattern) to a depth of 20m. The drains are assumed not to be penetrating the clay layer (drains closed at the bottom). Now compare a case where smear and well resistance are neglected with a case where smear and/or resistance are taken into account. In the latter case, the smeared zone is assumed to have a radius equal to two times the drain radius, i.e. $s = 2$, and a permeability of half the original permeability of the undisturbed clay, i.e. $k_c/k_c' = 2$. The discharge capacity q_w is assumed equal to $10\text{m}^3/\text{yr}$ in one case and $15\text{m}^3/\text{yr}$ in another.

The average degrees of consolidation for the 20m thick clay layer, obtained in the three cases with respect to the effect of vertical drains only, are given in Table I on page 44.

It is obvious that the classical solution based on $q_w = \infty$ may lead to an over-optimistic design. Back-calculations for the purpose of finding "true" values of the coefficient of consolidation from field data may further be misleading if used in connection with the design of an installation where the drains have a different discharge capacity from that in the case studied.

As regards the nominal diameter of a band-shaped drain I feel that the comparison made in Fig. 5 in my Paper provides ample evidence of the fact that the nearest possible value to choose is that of a circular cylinder having the same circumference as that of the band-shaped drain. I cannot comprehend the kind of considerations lying behind the doubts expressed by Fellenius with regard to this fact. It might be that Fig. 5 is difficult to read and that he has not fully realised its significance. It is therefore given here in redrawn form as Fig. 5. As can be seen, the consolidation obtained for the band-shaped drain is very nearly the same as that obtained for the circular drain with a nominal diameter chosen in accordance with the given assumption.

Another point raised by Fellenius is that of the filter requirements. These can easily be analysed by means of eqn. *6*.

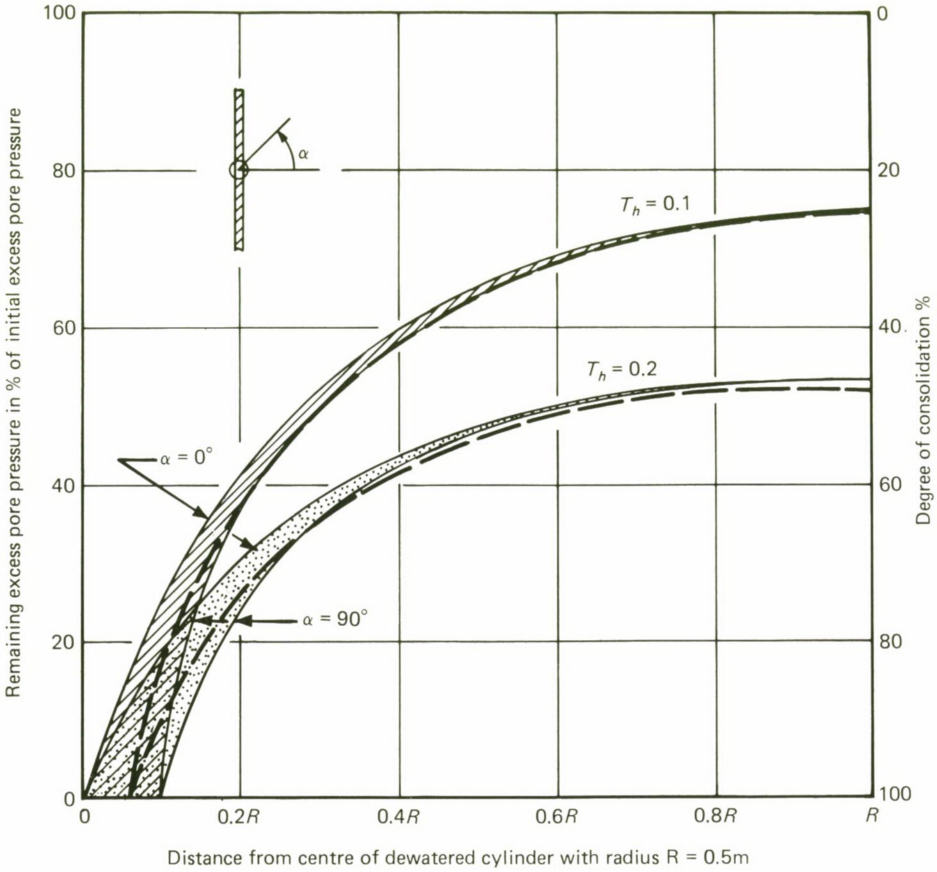

Fig. 5. Remaining excess pore water pressure and degree of consolidation at time factors $T_h = 0.1$ and 0.2 as functions of radius vector. Shadowed areas represent 4 × 100mm band-shaped drain (the possible variation with change in direction of radius vector) while broken lines represent circular drain, 66mm in diameter. Time factor $T_h = c_h t/D^2$. $D = 1m$

We find that the required permeability of the filter depends on

(*i*) the discharge capacity of the drain,
(*ii*) the drain diameter,
(*iii*) the thickness of the filter, and
(*iv*) the characteristics of the surrounding soil

Assuming that we have a discharge capacity of $q_w = 15m^3/yr$ (determined by the permeability of the drain core), a filter thickness of 0.2mm and a drain diameter of 0.066m, we find that the permeability of the filter need not be higher than 0.01m/yr (3×10^{-10} m/s), whatever the type of soil — silt or clay — in which the drains are placed. The explanation of this is what may be called "the corridor effect". If a corridor (the drain core) is filled with people (water) on their way out, one cannot cram more people into the corridor by opening more entrance doors (by increasing the filter permeability).

It is true, as indicated by Fellenius, that a low filter permeability may lead to a rise of water level in the drain and thus to a back pressure. However, this phenomenon seems to be of importance only in the beginning of the consolidation process when the rate of water flow into the drains is highest.

A more detailed explanation of how different drain characteristics will affect the process of consolidation has been published in the Proceedings of the Xth ICSMFE in Stockholm in 1981.

Fellenius takes me to task for not giving credit to Mr. Wager but only to Mr. Kjellman, who was the inventor of the "cardboard wick" — the first prefabricated drain on the market. It was unintentional on my part to deny or undervalue Wager's contribution to the development of new drains. But I see no reason why, in the event that Wager's name had been mentioned, the inventors of the other drains shown in Fig. 1 in my Paper should not also be credited at the same time — inventors whose names are unknown to me.

In response to Mr. Brodeur, like Dr. Fellenius, he is claiming that seams of silt and sand, as well as pockets of coarse material, necessitate a high permeability of the filter. The answer to this statement has been given already in my earlier comments.

I agree with Brodeur in his view of filter requirements with regard to practical aspects, but I do not agree with his conclusions regarding the filter paper. If his experience of paper as a filter material has proved bad, it probably derives from teething troubles with the early products. The filter paper utilised today in, for example, Geodrain has both high wet strength and a high enough filter permeability, about 1m/yr which is far higher than required. No difficulties of the kind indicated by Mr. Brodeur have been encountered in practice within my knowledge although about 8 million metres of paper filter drains have been installed. Geodrains have been successfully installed to depths of 30-40m without any negative effect on their efficiency (cf. Choa *et al.*, 1979 and Mongilardi & Torstensson, 1977).

Mr. Brodeur emphasises that he trusts in the experience of Mr. Oleg Wager, the inventor of Geodrain and Alidrain, regarding "limitations of early type drains" and the new important "concepts of larger free surface area and greater free volume". But only through extensive research on different kinds of prefabricated drains and careful parameter studies — and not through the use of imagination, however valuable this may be — is it possible to gain knowledge and experience. Very extensive research on prefabricated drains, probably the most comprehensive in the whole world, has been in progress for the last 10 years at the Geotechnical Department of Chalmers Tekniska Högskola, Gothenburg, on behalf of Terrafigo. This research includes all prefabricated drains available on the market, and the points raised in my Paper and in my present answers have been verified by a considerable number of field and laboratory tests. Therefore, as Mr. Brodeur's statement regarding the observed significance of the new concepts mentioned is not in agreement with our findings, it would be most interesting to see and examine the results of at least one, or hopefully two field tests, or laboratory experiments, in support of this statement.

Finally I should like to make a correction to eqn. *6* in my original Paper. The last term of this equation is unfortunately erroneous and should read:

$$\pi z(2l - z)\,\frac{k_c}{q_w}\left(1 - \frac{1}{n^2}\right)$$

instead of

$$\pi z(2l - z)\,\frac{k_c}{q_w}\left\{1 - \frac{k_c/k_c' - 1}{(k_c/k_c')\ (n/s)^2}\right\}$$

Thus, the last term can be further simplified to

$$\pi z(2l - z)\,\frac{k_c}{q_w}$$

However, the difference in the result given by slightly erroneous version of eqn. *6* and the corrected one is negligeable.

TABLE I

Time of consolidation	Discharge capacity q_w m³/yr					
	10		15		∞	
yrs	smear	no smear	smear	no smear	smear	no smear
0.5	20	23	23	27	33	42
1	36	40	40	51	55	66
2	58	63	64	78	80	88
4	82	85	89	93	96	99

References

Holtz, R. & Broms, B. (1972): "Long-term loading tests at Ska-Edeby, Sweden", Proc. Spec. Conf. on Performance of Earth-Supported Structures, June 11-14, Vol. 1, Purdue Univ., Lafayette, Ind.

Choa, V., Karunaratne, G. P., Ramaswamy, S. D., Vijiaratnam A. & Lee, S. L. (1979): "Pilot test for soil stabilisation at Changi Airport", Proc. of 6th Asian Regional Conference on SMFE in Singapore, Vol. 1, pp. 141-143.

Mongilardi E. & Torstensson, B. A. (1977): "800 000 metres of Geodrain in the Porto Tolle thermoelectric power plant project". Geotechnical Report from Terrafigo.

Consolidation of clay by band-shaped prefabricated drains

by SVEN HANSBO*

IN FOUNDATION ENGINEERING, consolidation settlement of clay and mud often creates serious problems. Significant consolidation settlement occurs when, for some reason, the preconsolidation pressure of the subsoil, representing the past maximum effective stress, is exceeded. When the stresses σ' in the subsoil exceed the preconsolidation pressure σ_c' the soil skeleton will break down (internal shear failure) and the stress increment above σ_c' will have to be carried by excess porewater pressure (in water saturated soils) or by a combination of excess porewater and excess pore gas pressures (in non-saturated soils). Porewater (and dissolved pore gas) will thereby be squeezed out of the soil until the soil skeleton is again able to carry the load.

This consolidation process is governed by the rate of excess pore pressure dissipation, i.e. by the coefficient of consolidation c_v of the soil and of the thickness of the consolidating layer. In a case where the clay or mud layer is homogeneous (having no horizontal continuous highly permeable seams or layers) and the width of the load placed on the layer is large in comparison with the thickness of the layer, the porewater is squeezed out mainly in the vertical direction. In such a case the time during which consolidation settlement will occur is often very long — for a 10m homogeneous clay later, drained at the top and bottom, some 50 to 100 years depending upon the magnitude of the coefficient of consolidation of the soil. If the thickness of the homogeneous layer is doubled consolidation time will be increased four-fold.

To reduce consolidation time it is obviously necessary to shorten the length of the flow paths. One way of doing this is to install vertical drains of high permeability. Thereby porewater can also escape in the horizontal direction towards the drains and flow freely along the drains vertically to a drainage blanket placed on the soil surface or to other highly permeable layers deeper down in the soil.

Drain types

The best-known type of drain in foundation engineering is the sand drain. It is probably less well known that prefabricated drains were introduced into the field of geotechnical engineering in 1937, almost simultaneously with sand drains (Hansbo, 1977). First on the market was the Kjellman cardboard wick, a band-shaped drain, 3.5mm by 100mm, made up of two cardboard strips, with ten longitudinal grooves, glued together so that the grooves formed longitudinal channels (Fig. 1*a*). A special machine for its installation was constructed in 1939.

Subsequently several types of band-shaped drains were developed all over the world and today the number of new prefab drains appearing on the market is rapidly increasing. Some of these are presented in Figs. 1*b-g*.

The shape (width and thickness) of the new prefab drains is very nearly equal to that of the prototype — the cardboard wick. Only the Colbond drain has a significantly larger width than the others. The drains usually have a core of plastic material with a filter sleeve of paper or some other fibrous material, usually of plastic. A system of vertical channels between the core and the filter sleeve is secured in various ways. The Colbond drain, however, is made of the same material throughout — a non-woven fabric with a lower boundary permeability.

Engineers who have worked only with vertical cylindrical drains of conventional type, i.e. sand drains, may be unfamiliar with the band-shaped drain and be unaware of how to apply the knowledge obtained from their previous experience of vertical drain installations. It therefore seems necessary to make clear how to design a vertical drain installation with prefab band-shaped drains and to elucidate the difference, if any, between these drains and the more well-known cylindrical sand drains.

Design considerations

Drain spacing

The theoretical calculation of the maximum drain spacing required to obtain a desired result is based on the classical assumption that each drain has a zone of influence represented by a circular cylindrical soil column of the same length as the drain and containing that volume of soil from which water can be assumed to be squeezed (or sucked) into the drain in question. It can then be readily found that the diameter D of the dewatered cylinder varies from 1.05 times the spacing when the drains are placed in an equilateral triangle grid to 1.13 times the drain spacing when they are placed in a square grid.

Another basic assumption which considerably facilitates the theoretical calculation is that, during consolidation, horizontal sections remain horizontal (equal strain theory). The difference between the results thus obtained and the results of — as is often believed — the more correct assumption of a free development of strains in the clay between the drains (free strain theory) is negligible (Barron, 1944). Moreover, settlement observations in the field strongly support the assumption of equal vertical strains (*cf.* Holtz & Holm, 1972). In the classical solution it is further assumed that the permeability of the drain is infinite in comparison with that of the clay and that Darcy's law is valid.

For a saturated soil we then obtain

$$\bar{U}_h = 1 - e^{-8T_h/\mu} \qquad \ldots (1)$$

$$\text{or} \quad t = \frac{D^2\mu}{8c_h} \ln \frac{1}{1 - \bar{U}_h} \qquad \ldots (1a)$$

where

$\bar{U}_h$ = average degree of consolidation taking into account only the effect of the vertical drains

$$\mu = \frac{n^2}{n^2 - 1}\left[\ln(n) - \frac{3}{4} + \frac{1}{n^2}\left(1 - \frac{1}{4n^2}\right)\right]$$
$$\approx \frac{n^2}{n^2 - 1}\left[1\text{n}(n) - 0.75 + n^{-2}\right]$$

$T_h = c_h t/D^2$,

t = time of consolidation,

$c_h = \dfrac{k_h M}{g\rho_w}$ = coefficient of consolidation in horizontal porewater flow,

$M = 1/m_v$ = compression modulus,

k_h = permeability in horizontal direction,

ρ_w = density of water,

g = acceleration of gravity,

$n = D/d$,

D = diameter of dewatered soil cylinder, and

d = diameter of drain.

The maximum drain spacing for different drain diameters required to obtain a certain average degree of consolidation according to eqn. *1* can be read from Fig. 2.

As has been shown in several investigations (e.g. Hansbo, 1960) Darcy's law is sometimes invalidated at small hydraulic gradients prevailing in practice in drained areas. Thus, the results of permeability tests in the laboratory and of full-scale consolidation tests at Ska-Edeby, Sweden, showed that the relation between porewater flow v and hydraulic gradient i in this case followed the exponential law $v = Ki^n$ where K = the coefficient of permeability in non-Darcian flow. A new solution to the "equal strain" consolidation theory based on this exponential law was presented by the author (Hansbo, 1960).

The best agreement between the full-scale test results at Ska-Edeby and this new theory was obtained for the exponent value n = 1.5 (Hansbo, 1960; Holtz & Broms, 1972). For this value, the new theory gives

$$t = \frac{\alpha}{\lambda} D^2 \sqrt{Dg_{\rho_w}/\Delta\bar{u}_0}\left(\frac{1}{\sqrt{1 - \bar{U}_h}} - 1\right) \qquad \ldots (2)$$

where

$\Delta\bar{u}_0$ = average excess pore pressure at $t = 0$

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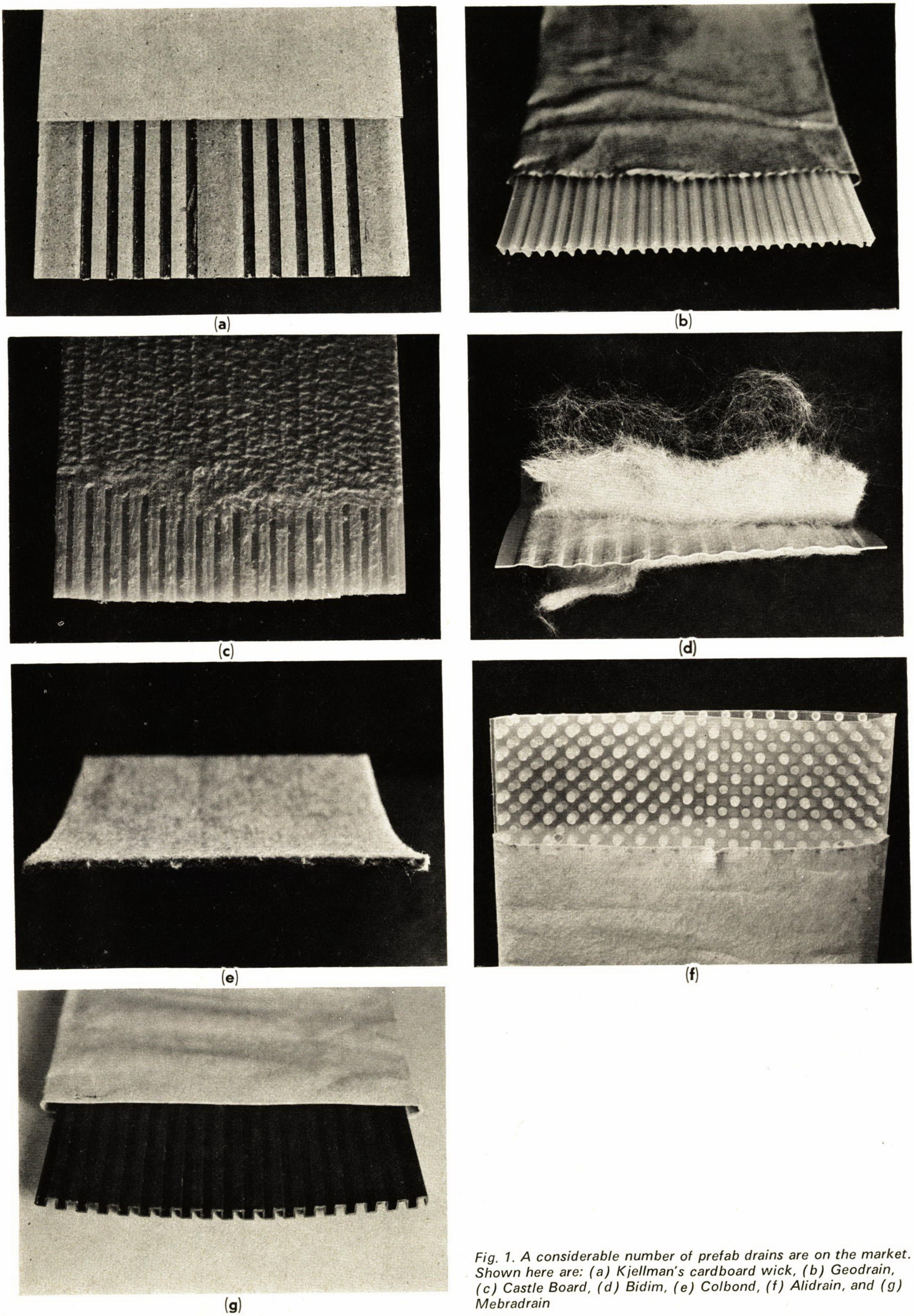


Fig. 1. A considerable number of prefab drains are on the market. Shown here are: (a) Kjellman's cardboard wick, (b) Geodrain, (c) Castle Board, (d) Bidim, (e) Colbond, (f) Alidrain, and (g) Mebradrain

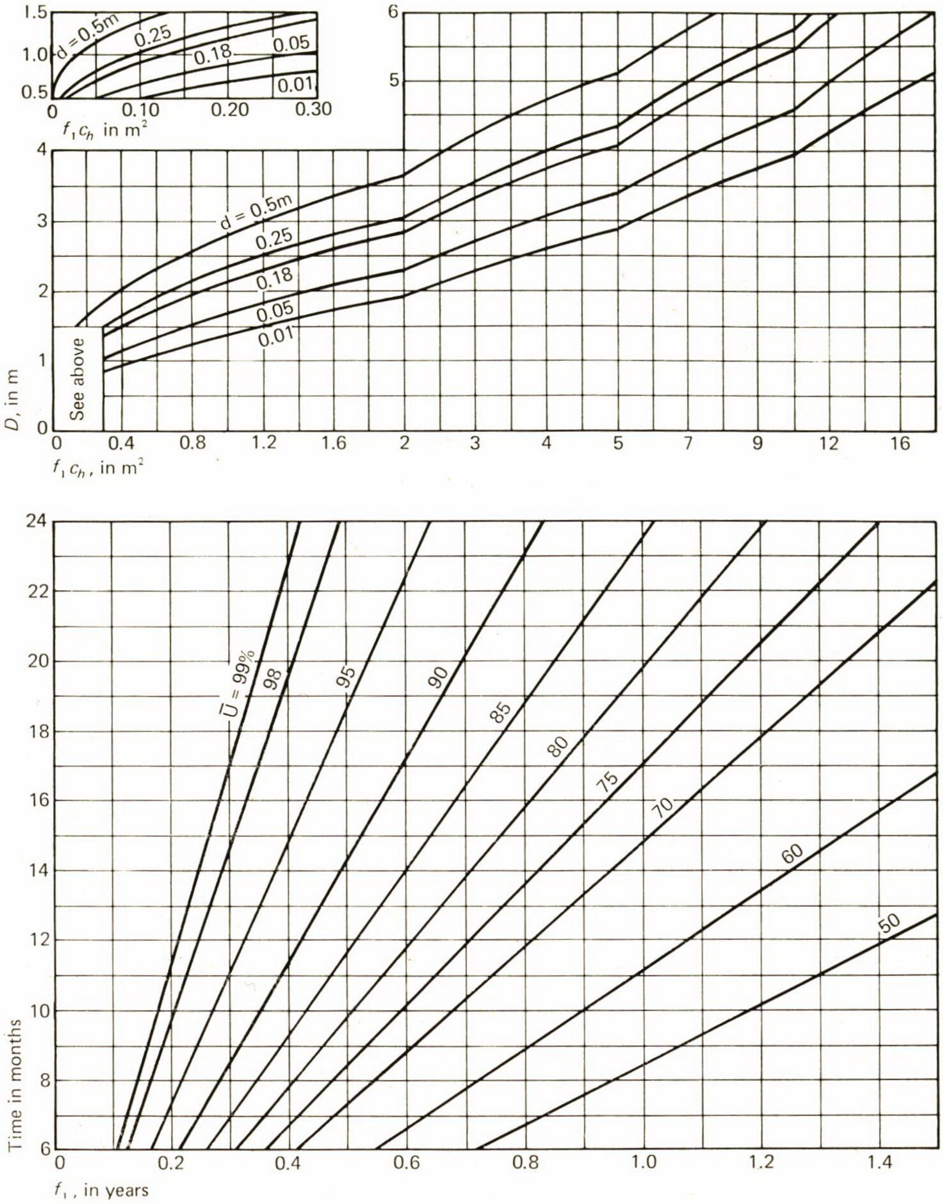


Fig. 2. Graph for design of vertical drain installations based on eqn. (1).
Parameter $f_1 = D^2\mu/8c_h$
Example: What drain spacing is required to reach 90% average degree of consolidation after 1 year? Homogeneous clay with $c_h = 0.5m^2/y$. Drain diameter 0.05m.
Solution: Lower diagram gives $f_1 = 0.44y$ whence $f_1c_h = 0.22m^2$. Upper diagram gives $D = 0.9m$, from which drain spacing $= 0.8m$

α = function of D/d (Fig. 3),

$$\lambda = \frac{KM}{g\rho_w} = \text{coefficient of consolidation in horizontal non-Darcian porewater flow and}$$

M, g and ρ_w are as noted above.

The coefficient of consolidation λ can be assumed to be approximately equal to c_v determined by the oedometer test.

In eqn. 2 the magnitude of the consolidation load (i.e. the instantaneous excess pore pressure $\Delta\bar{u}_0$ due to loading) has an influence on the time of consolidation: the higher the load, the shorter the time of consolidation. This is encountered in many cases in practice. Furthermore, the process of consolidation obtained from eqn. 2 is more rapid at the beginning, a fact which also agrees in many cases with practical experience, particularly on a site with soft highly plastic clay.

The maximum spacing of drains, 50mm in diameter, required to obtain a certain average degree of consolidation according to eqn. 2 can be read from Fig. 4.

The difference between eqns. *1* and *2*, in view of the drain spacing required to obtain a certain degree of consolidation at a certain time *t*, is fairly unimportant in most practical cases.

Equivalent diameter *d* of a band-shaped drain

In the theoretical calculation it is presumed that the drain is a circular cylinder with the diameter *d*. When dealing with a band-shaped drain we therefore have to assume a *d* value that will produce the same effect as the band-shaped drain in question. This question was treated by Kjellman (1948) who stated that "the draining effect of a drain depends to a great extent upon the circumference of its cross-section, but very little upon its cross-sectional area" and that "certain considerations show that the cardboard wick is as effective as a circular drain with a 1in radius".

Kjellman's assumption has been verified by finite element analysis (Runesson, Tägnfors & Wiberg, 1977). The result of this analysis is shown in Fig. 5. Obviously, the equivalent diameter of a band-shaped drain with width *b* and thickness *t* can be expressed by

$$d = \frac{2(b+t)}{\pi} \qquad \dots (3)$$

This means that the equivalent diameter is 66mm in the case of Geodrain ($b =$ 100mm; $t =$ 4mm), 68mm in the case of Alidrain ($b =$ 100mm; $t =$ 7mm) and 98mm in the case of Colbond ($b =$ 300mm; $t =$ 4mm). Larger numerical values of the equivalent diameter than those determined by eqn. *3* cannot be used in the design since the discharge capacity of the drains has been assumed to be infinitely large (no well resistance).

In practice, the drain spacing is seldom below 0.8m. For prefab drains we thus find $n > 12$ (>8 in the case of Colbond). This justifies a further simplification of μ in eqn. *1*

$$\mu \approx \ln(n) - 0.75 \qquad \dots (4)$$

Well resistance

In reality, a drain with infinite permeability in the longitudinal direction (infinite discharge capacity) — one of the basic assumptions in the deduction of eqns. *1* and *2* — does not exist. The drain has a certain well resistance and the well resistance might be so high that the results obtained by eqns. *1* and *2* would lead to an over optimistic design of the drain installation.

If it is assumed that the discharge capacity of the drain (the well) is q_w and that the permeability of the soil is k_c (Darcy's law is assumed to be valid), μ in eqn. *1* should be replaced by

$$\mu_r \approx \ln(n) - 0.75 + \pi z\,(2l - z)k_c/q_w \quad \dots (5)$$

where

l = length of the drain when open at one end only (half length of the drain when open at both ends)
Z = distance from open end of drain ($0 \leqslant z \leqslant 2l$)
n and d as before

Obviously, the relative influence of well resistance depends on the drain diameter and drain spacing (the *n* value) and on the length of the drains and the ratio k_c/q_w. For a typical band-shaped drain, 20m in length and closed at the bottom, we find for example that well resistance cannot usually be ignored when $q_w/k_c < 3\,000\text{m}^2$.

In the case of prefab drains in particular, the requirements placed on the drain with regard to discharge capacity have to be taken into account as many of the drains appearing on the market can be suspected of having a considerable well resistance.

The best way of finding out if the discharge capacity is high enough is, of course, to make full-scale in situ tests. If laboratory tests are used the well resistance of the drain has to be investigated under conditions similar to those in the field.

Effect of disturbance

The insertion of the mandrel causes more or less severe remoulding of the subsoil, mainly in the immediate vicinity of the mandrel but sometimes also at a fairly large distance from it (Hansbo, 1960). As remoulding leads to a decrease in the coefficient of consolidation and thereby to a delay in the consolidation process it has to be considered in the theoretical calculation. This can be

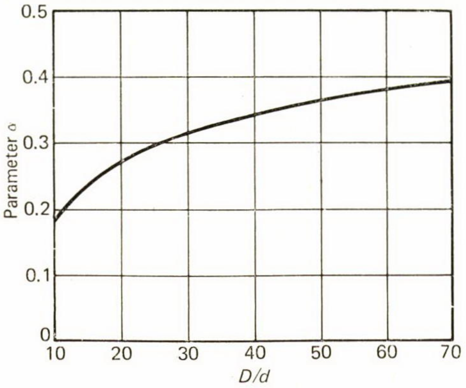


Fig. 3. Parameter α vs. D/d in eqn. (2)

done either by introducing a zone of smear (Barron, 1948) with reduced permeability or by simply assuming a lower overall value of the coefficient of consolidation or, alternatively, by using a reduced value of the equivalent diameter. With the same basic assumptions as stated previously and assuming a zone of smear with diameter d_s, we find that μ in eqn. *1* should be replaced by

$$\mu_s \approx \ln\left(\frac{n}{s}\right) + \frac{k_c}{k_c'}\ln(s) - 0.75$$

$$+ \pi z(2l-z)\frac{k_c}{q_w}\left\{1 - \frac{k_c/k_c' - 1}{(k_c/k_c')\ (n/s)^2}\right\} \dots \quad (6)$$

in which

$s = d_s/d$,

d_s = diameter of disturbed zone,

k_c' = permeability of disturbed zone and

d, k_c, q_w, z, l and n as before.

According to the literature, a common value of s is about 1.5-3 whereas the ratio k_c/k_c' is more or less open to discussion. For a band-shaped drain s is mainly dependent on the type and size of the mandrel and probably also on the method of installation (dynamic or static). The larger the circumference of the mandrel, the larger the nominal value of s. The mandrel ought therefore to be formed after the drain with as close clearance fit as possible.

Instead of assuming a zone of disturbance with diameter d_s, either an overall reduction of the coefficient of consolidation c_h can be made or the drain diameter d can be given a smaller nominal value in eqns. *1* and 2. By doing this the design diagrams, Figs. 2 and 4, can be utilised as they are. The best general correspondence with eqn. *6* is obtained by the last-mentioned method.

Filter requirements

The requirements to be placed on the filter sleeve are as follows:

(*1*) the permeability of the filter should not be considerably less than the permeability of the soil n which the drains are placed,

(*2*) the filter should retain fine soil particles. Otherwise the channels between the filter sleeve and the core might eventually be filled with soil and get clogged,

(*3*) the filter should be strong enough not to get completely squeezed into the channel system of the core by high lateral soil pressure,

(*4*) the filter should be strong enough not to break during installation, and

(*5*) the filter should not deteriorate with time if this would endanger the discharge capacity of the drain.

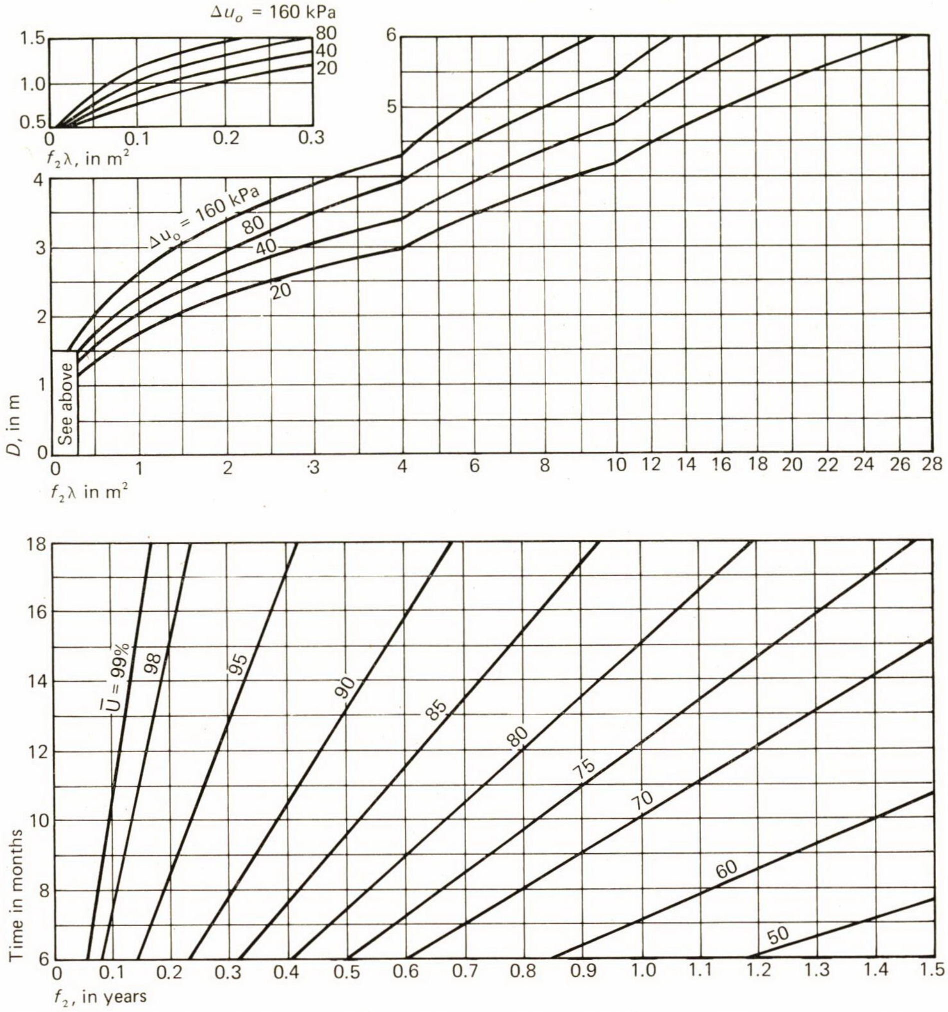


Fig. 4. Graph for design of the spacing of 50mm drains according to eqn. (2). Parameter $f_2 = \dfrac{\alpha D^2}{\lambda}\sqrt{Dg\rho_w/\Delta u_o}$. *Drain spacing dependent on loading conditions.*

Example: What drain spacing is required to reach 90% average degree of consolidation after 1 year? Homogeneous clay with $\lambda = 0.2m^2/y$. *Load* $= 80kN/m^2$

Solution: Lower diagram gives $f_2 = 0.46y$ *whence* $f_2\lambda = 0.09m^2$. *Upper diagram gives* $D = 1.0m$ *from which drain spacing* $= 0.9m$

If the filter is very thick and of low permeability it may have some influence on the nominal diameter of the drain. Thus, a filter with the same permeability as the surrounding soil will of course have to be considered as being part of the soil rather than part of the drain.

Free surface area — free volume

The new concepts of "free surface area" and "free volume" have appeared and been used in connection with prefabricated drains. A drain with a larger free surface area and a larger free volume is claimed to be superior to a drain of equal shape with smaller free surface area and smaller free volume. Therefore, it is claimed that a larger drain spacing can be chosen.

One assumption behind this is that the transverse flow of water from the soil into the drain takes place more freely in the case of a large free surface area. Another assumption is that the longitudinal flow of water within the drain itself also occurs more freely with a large free volume.

But it is clear from the equations for consolidation due to vertical drains that an increase of inlet capacity (free surface area) or of discharge capacity (free volume) above certain limits is of no value whatsoever. As long as the permeability of the filter has the same order of magnitude as that of the soil, the inlet capacity is sufficiently high and an increase will have no influence on transverse flow.

The discharge capacity requirements were formulated by eqns. *1* and *5*. Even in this case an increase of discharge capacity is valueless above that needed for avoiding perceptible influence of well resistance on the consolidation process.

Need for vertical drains

When considering the possible need for vertical drains, a question of the utmost importance has to be answered, namely if loading would increase the stresses in the soil to values above the preconsolidation pressure. It is amazing how often the installation of drains is contemplated without any knowledge of the preconsolidation pressure of the subsoil in question.

Doubtless, the ineffectiveness of vertical drains sometimes experienced can be explained by the fact that the preconsolidation pressure is not exceeded due to loading. In such a case, the rate of consolidation is almost the same, drains or no drains, and the only effect obtained might be an increase of the settlement due to the disturbance caused by drain installation.

Drains should not therefore be used unless it has been established that the stresses in the soil due to loading will exceed the preconsolidation pressure. The

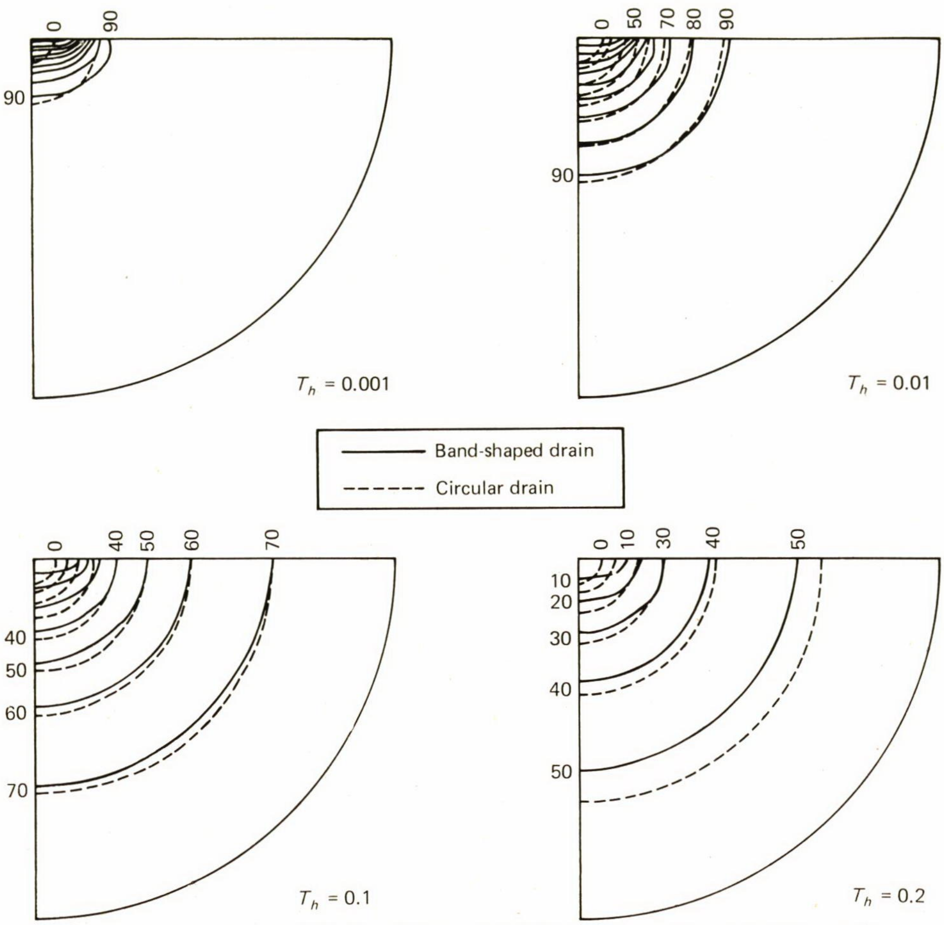


Fig. 5. Comparison of consolidation effects (remaining excess pore pressure Δu after different times of consolidation in % of initial excess pore pressure Δu_0) caused by a 100mm x 4mm band-shaped drain and a circular drain with equivalent circumference (d = 66mm). Time factor $T_h = c_h t/D^2$. D = 1m

CRS and the CGT oedometer tests seem to offer today the best existing ways of determining the preconsolidation pressure (*cf.* Sällfors, 1975). These testing procedures also seem best for the determination of the c_v value.

The compression modulus $M = 1/m_v$ found from the inclination of the virgin curve as obtained in the oedometer test might have to be modified with respect to the disturbance effects of the drain installation. The more disturbed the samples are which have been selected for the oedometer test, the greater the need for such a modification. This fact is not always realised although it is well known that the inclination of the virgin curve decreases — implying that M increases — with increasing sample disturbance. The value of M therefore has to be more or less reduced. One method of correction which has proved to be reasonably adequate for displacement type sand drains, 0.18m in diameter, has been suggested by the author (Hansbo, 1960).

Installation methods

Several methods of installation have been used. The easiest way for a contractor is, of course, to equip a piling machine with some suitable mandrel. Two basically different methods of installation can be recognised — dynamic and static.

In the former case, the mandrel is driven into the soil with the aid of either a vibrating hammer (Fig. 6) or, less often, a conventional drop hammer. In the latter case, the mandrel is pushed into the soil by means of a static force (Figs. 7 & 8).

It is not yet known which of the methods is best with regard to consolidation. However, the dynamic methods seem to create a higher installation excess porewater pressure and therefore may affect the stability of nearby slopes.

Prediction and observation

Difficulties of prediction

In addition to the difficulties of determining the compression modulus M (as discussed above) the main concern in the design of a vertical drain installation is to find the correct value of the coefficient of consolidation with horizontal porewater flow c_h. This usually has a higher numerical value than the coefficient of consolidation c_v determined by oedometer tests in the laboratory with porewater escape in the vertical direction. The ratio c_h/c_v is often 2-5.

Another important factor which has to be considered is the possible existence of sand and silt seams, or pockets, in the soil. Such seams or pockets can be mapped, e.g. by means of pore pressure sounding (Torstensson, 1975) but it is impossible to distinguish continuous horizontal seams from isolated pockets. Continuous seams would act as horizontal planes of drainage and thus contribute considerably to a quicker rate of consolidation. The influence of such seams on the consolidation process cannot be forecast by oedometer tests on a laboratory scale.

The magnitude and effect of disturbance caused by the drain installation as well as the magnitude of well resistance (eqns. *5* & *6*) are also difficult to estimate. As shown previously, these factors may have a considerable delaying influence on the rate of consolidation.

There is no doubt that the correct design of a drain installation requires considerable experience of the effect of vertical drain installations on the soil properties and a good knowledge of the theoretical background to the consolidation theory.

Checking of rate of consolidation

The methods used for checking the degree of consolidation generally consist of measuring the settlement and/or the excess pore pressure dissipation.

In the case of settlement studies, the final consolidation settlement has to be known and therefore the accuracy of the method is completely dependent on how well this settlement can be calculated in advance. The most useful information is obtained if the settlement is observed at different depths in the soil, which can be done, for example, by the bellows settlement hose developed at SGI (Wager, 1973).

In the case of pore pressure measurements, the observed excess pore pressure Δu will be a function of the distance to the nearest drain. It is obvious from Barron's "free strain" analysis that the best estimate of the average degree of consolidation is obtained by studying the excess pore pressure at a distance from a drain of one quarter of the drain spacing (half way between the drain and the outer boundary of the dewatered cylinder, *cf.* Barron, 1947). The "error" in the measurements is at its greatest about 10% (in the beginning of the consolidation process) and decreases with the time of consolidation. If the piezometer is placed closer to the drain the error is greater and increases rapidly with decreasing distance.

The results of the two above-mentioned checking methods are not directly comparable as they do not give equal values of $\bar{U}$ (see, for example, Hansbo, 1960 and Holtz & Broms, 1972). $\bar{U}$ obtained from settlement studies in clays of very low permeability is generally higher than from pore pressure studies. Secondary compression during the hydrodynamic primary consolidation period might furnish one explanation to this discrepancy.

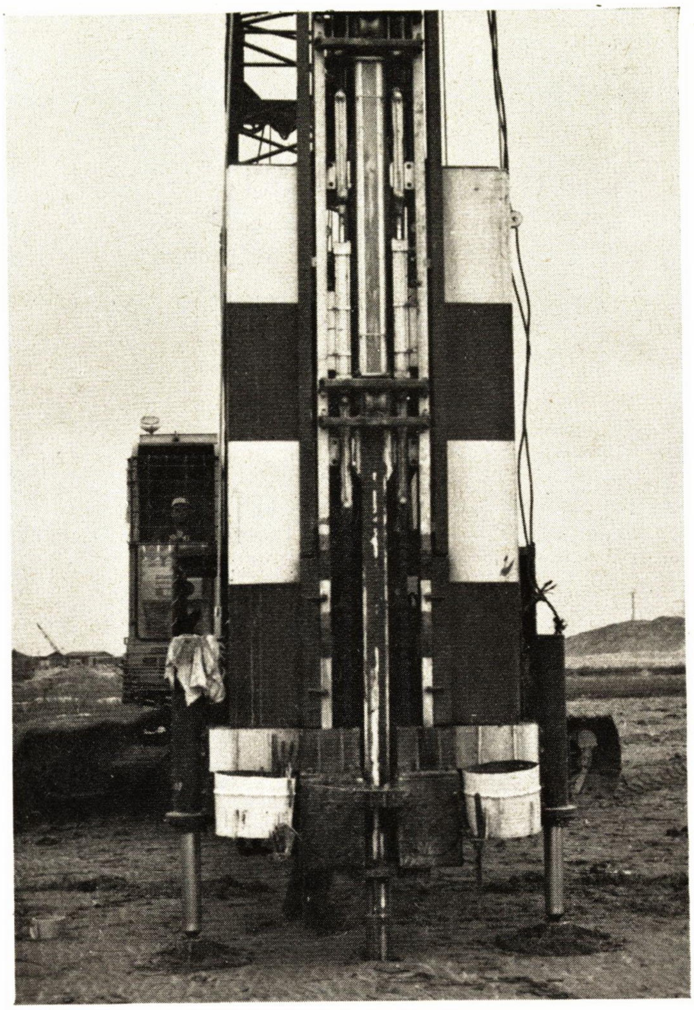
Another apparent anomaly is that excess pore pressure may remain in the soil under a load embankment, indicating that consolidation is still in progress although settlement has completely ceased (Holtz & Broms, 1972; Hansbo, 1977). This has long been a major concern and hard to explain. The most probable explanation in the opinion of this author is that the steady state groundwater level within the filled area in general will be higher than before the fill was placed.

Thus, the groundwater level, observed for example in its natural state, seems to have a topography fairly similar to that of the ground surface.

Case records

A number of test sites have been monitored in order to check the correlation between prediction and actual behaviour of load embankments on clay provided with prefab drains (mainly of the Geodrain type) and sand drains. Prediction (or back calculation) has been based on the assumption that the Geodrain has a 50mm diameter. In assuming this, the possible effect of disturbance and/or well resistance is taken into account. The results of most of these investigations have been published, for example in a number of issues of "Geotechnical Report from

Fig. 6. *Installation of Geodrain by means of a vibrating hammer. Contractor, Cofra* (photo, Dienst Publieke Werken, Amsterdam)



(a)

(b)

Fig. 7. (a) Installation of Geodrain by a static force. Contractor, Rodio. (b) The drain which is placed inside the mandrel has a tip anchor; this is a thin steel plate which easily folds when the mandrel is driven into the soil. The anchor protects the mandrel from becoming filled with soil when pushed into the ground

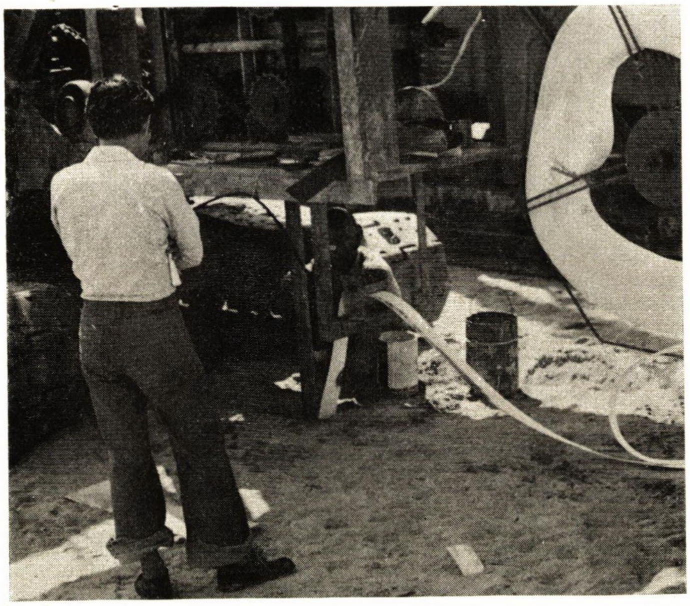


Fig. 8. Installation of Castle Board drain by a static force. In this case, the drain is placed outside the mandrel

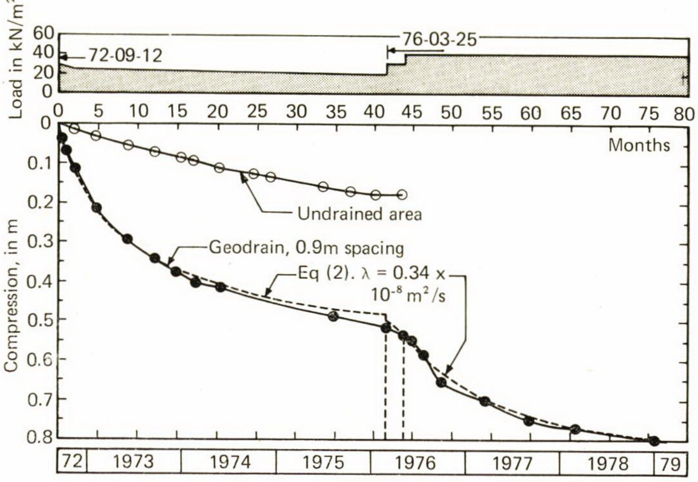


Fig. 9. Long-term compression of 5m thick clay layer, between 2.5m and 7.5m depths, at Ska-Edeby, Sweden. Geodrain with 0.9m spacing. Test area with sand fill, 30m in base diameter. Additional fill 15m in base diameter

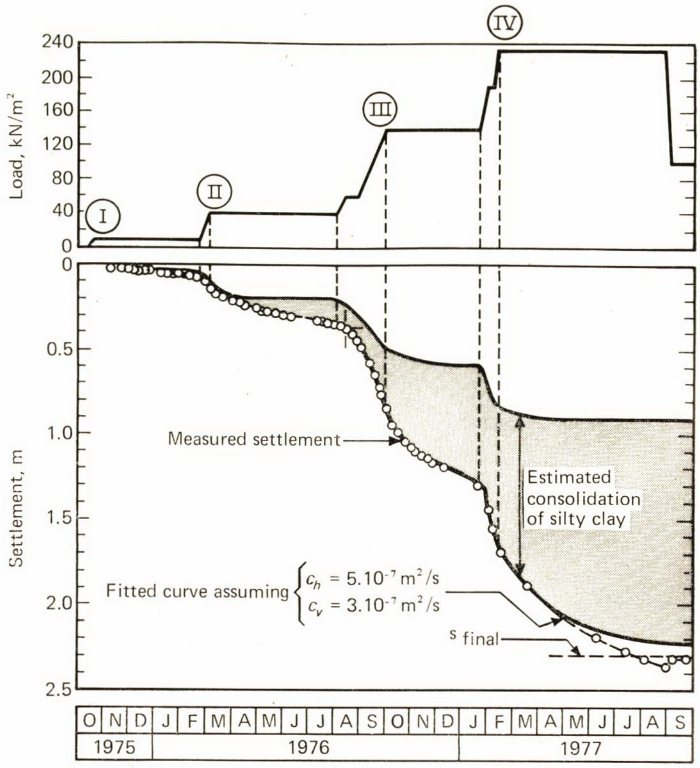


Fig. 10 (right). Long-term compression, between 8m and 20m depths, of a 21m thick silty clay layer, at Porto Tolle, Italy. Geodrain with 3m spacing. Test area with sand fill, 150m in base diameter. Curve fitting and theoretical prediction of s_{final} carried out on the basis of data obtained up to early 1977 (Mongilardi & Torstensson, 1977). Embankment load decreased to about 100kN/m² in Sept. 1977; thereafter, a slight heave has been measured

Terrafigo", Stockholm. Other investigations, as yet unfinished, will be reported in the near future.

A particularly illuminating result is given in Fig. 9. This represents the longest observation carried out in order to study the behaviour of prefab drains and it shows that the drain in question, the Geodrain, has functioned as expected for some 4-5 years after installation. Another example is given in Fig. 10.

To sum up, all the tests carried out up to the present show good agreement between theory and practice if reasonable values of c_h (or λ) are adopted. The nominal value of d for an efficient band-shaped drain, 100mm in width and 3-7mm in thickness, to be used in the design of a drain installation can be chosen — it seems — as 50mm. A smaller nominal value may have to be used in a case where the drain is not fully efficient due to poor discharge capacity.

Prefab drains in the future

Innovations to be expected

The number of prefab drains is steadily increasing and new innovations, if for no other reason than to circumvent existing patents, are frequently appearing. The most important requirements to be placed on these drains are that they should not become blocked with time and that their discharge capacity be sufficiently high.

To prove that these requirements are met, full-scale test embankments on clay provided with the type of drain in question should be monitored. Otherwise, prefab drains that could prove unsuitable in practice might be utilised, creating a bad reputation that could also cast a shadow over other types of prefab drains of good performance.

Use of vacuum

Instead of a load embankment causing consolidation due to excess pore pressure in the clay between the drains, vacuum can be applied in the drains. As a result the suction in the drains will cause a hydraulic gradient in the clay, in a direction towards the drains, in a similar way as in the case of an embankment load. If full vacuum could be attained the effect would obviously correspond to a surcharge of 100kN/m². However, about 80% vacuum seems to represent the present limit.

Vacuum has the advantage over load embankments in that no stability problems arise. For a successful application, the drains have to be in a homogeneous, impermeable soil. If they are in contact with highly permeable layers, the possibility of attaining the desired degree of vacuum is limited.

In principle, a result equal to that of the vacuum method is obtained by lowering the water pressure in a sand layer in the clay which is penetrated by the drains.

Vertical drains in combination with heavy tamping

As pointed out in an earlier Paper by the author in this journal (Hansbo, 1978) a combination of heavy tamping and vertical drains can be useful in land reclamation where loose sand fill is placed on top of thick clay deposits. This combined method of soil stabilisation is, for example, being used in the construction of Changi Airport in Singapore (Fig. 11).

An interesting investigation of the dynamic consolidation of a soft, highly plastic clay deposit in Uddevalla, on the west coast of Sweden, was started up three years ago by the author. The results obtained up to now are given in Fig. 12. Two test areas were monitored, one without and one with prefab drains, of the Geodrain type, 14m in length and with 2m spacing. Heavy tamping was carried out with three drops of a 14t weight from a 25m height at points 5m apart. The rate of excess pore pressure dissipation in the undrained area shows that heavy tamping in itself caused an increase in the coefficient of consolidation c_v from about 0.5m²/y to 2.5m²/y. In the area equipped with Geodrains the excess pore pressure has nearly vanished after 920 days. The effect of the vertical drains is in agreement with theory provided that c_h is assumed equal to 2.5m²/y.

These results, and those obtained in Singapore (Choa *et al.*, 1978), indicate that heavy tamping and prefab drains which can stand the severe treatment of heavy tamping can be combined in order to stabilise, in a most economic and expedient way, reclaimed land in soft clay regions.

References

Barron, R. A. (1944): The influence of drain wells on the consolidation of fine-grained soils", Diss. Providence, US Engng. Office.

Barron, R. A. (1948): "Consolidation of fine-grained soils by drain wells", Trans. ASCE, Vol. **113**, Paper No. 2346.

Choa, G. P., Karunaratne, S. D., Vigiaratnam, A. & Lee, S. L. (1978): "Pilot test for soil stabilisation at Changi Airport" Submitted to the Asian Regional Conference on SMFE in Singapore, July 1979.

Hansbo, S. (1960): "Consolidation of clay, with special reference to influence of vertical sand drains", Thesis, Swedish Geot. Inst., Proc. No. 18.

Hansbo, S. (1977): "Geodrains in theory and practice", Geotechnical Report from Terrafigo, Stockholm.

Hansbo, S. (1978): "Dynamic consolidation of soil by a falling weight", Ground Engng, July, Vol. **11**, No. 5.

Holtz, R. & Broms, B. (1972): "Long-term loading tests at Ska-Edeby, Sweden", Proc. Spec. Conf. on Performance of Earth-Supported Structures, June 11-14, Vol. **1**, Purdue Univ., Lafayette, Ind.

Holtz, R. & Holm, G. (1973): "Excavation and sampling around some sand drains at Ska-Edeby, Sweden", Proc. Nordic Geot. Meeting in Trondheim, Aug. 24-26, 1972, Norw. Geot. Inst., Oslo.

Kjellman, W. (1937): "Redogörelse för Statens geotekniska instituts verksamhet under aren 1944-1948", Swedish Geot. Inst., Meddelande No. 2.

Kjellman, W. (1948): "Consolidation of fine-grained soils by drain wells", Transactions ASCE, Vol. **113**, Contribution to the discussion.

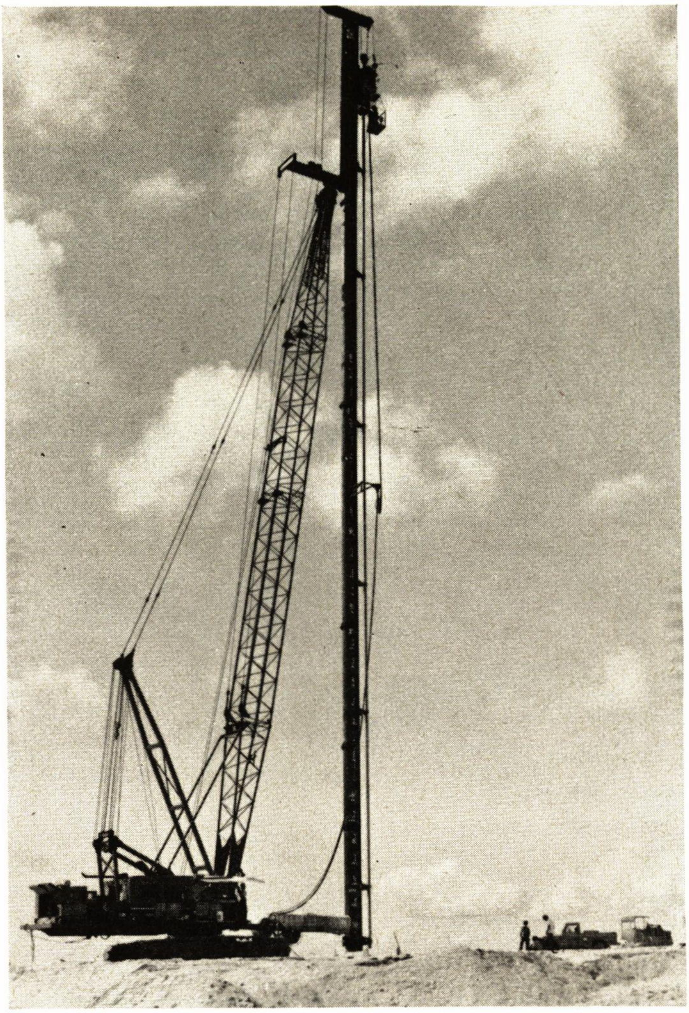
Mongilardi, H. & Torstensson, B. A. (1977): "800 000m of Geodrain in the Porto Tolle thermo-electric power plant project". Geotech. rept. from Terrafigo, Sweden.

Sällfors, G. (1975): "Preconsolidation pressure of soft high-plastic clays", Thesis, Division of Geotech. Engng., Chalmers University of Technology, Sweden.

Torstensson, B. A. (1975): "The pore pressure sounding instrument", ASCE Spec. Conf. on In-Situ Measurement of Soil Properties, Vol. **2**, Raleigh, USA.

Runesson, K., Tägnfors, H. & Wiberg, N-E. (1977): "Nagra geotekniska och geohydrologiska problem analyserade med finit elementmetod", Chalmers University of Technology, Division of Building Statics, Publ. 77:3.

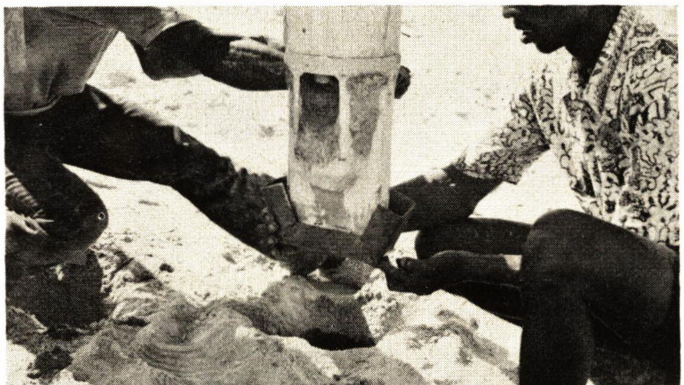
Wager, O. (1973): "Bälgsättningsmätare för mätning av vertikalrörelser i jord", Nordic Geot. Meeting in Trondheim, Aug. 24-26, 1972, Norw. Geot. Inst., Oslo.



(a)

(b)

Fig. 11. (a) Installation of Geodrain to a depth of 43m on reclaimed land in Singapore. Length of mandrel 50m. Contractor, TLM. (b) Buckling of the mandrel is prevented by moveable, closely spaced lateral supports. (c) Due to a hard surface layer of hydraulically placed sand a rugged, profiled drain anchor is being used



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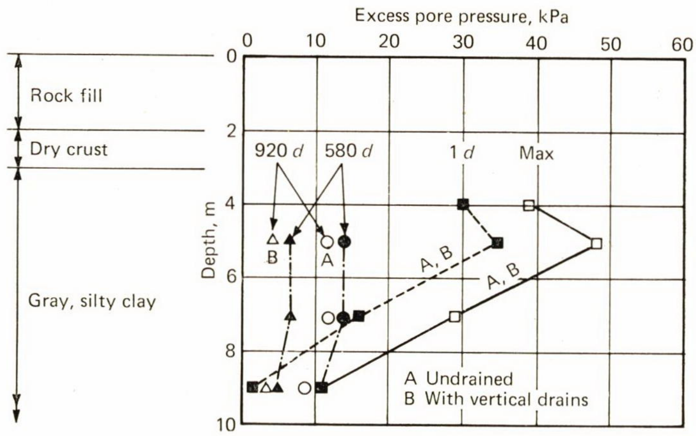

Fig. 12. Effect of dynamic consolidation on soft, highly plastic clay in Uddevalla, Sweden. This gives a comparison between excess pore pressure dissipation in a test area provided with Geodrains at 2m spacing, and excess pore pressure dissipation in another test area without drains