Six-storey Building on Soils Improved by Sand Drains

Bâtiment à Six Etages sur des Sols Améliorés par des Drains de Sable

A. CHALMERS BSc MICE CEng, Contracts Manager, Geotechnical Dept., Cementation Ground Engineering Ltd., Rickmansworth, UK

A.B. HARRIS CEng FI Struct E FWI, Partner, Bylander Waddell Partnership, Harrow, UK

SYNOPSIS At Ulverston, Cumbria (UK), a raft foundation for a heavy 6-storey steel frame structure was placed on 14m of weak cohesive soils which were improved within a short programme period by a combination of preloading and vertical drainage by Sandwicks. In the paper, the approaches to the design of the vertical drainage layout and preloading programme are described. The behaviour of the soils during preloading was monitored by instruments and the results are presented and compared with the predictions from the laboratory tests. Movements of the building have also been monitored for five years since the concrete raft was constructed.

INTRODUCTION

A steel framed industrial building has been erected for Glaxo Operations (UK) Ltd. at the fringe of an existing pharmaceutical factory complex at Ulverston, Cumbria, England. The building is sited some 30m from two existing single storey factory buildings and chemical plant. A first stage investigation borehole revealed very weak estuarine soils to about 14m, which are inferior to conditions in other parts of the complex. The proposed building site is located close to a former tidal inlet.

The use of ground treatment by sanddrains was examined and, coupled with the availability of a nearby source of waste slag for a preload embankment, a preliminary assessment showed that this ground improvement method for a raft foundation was feasible. It offered both a financial saving and an advantage over piling as a construction method, provided the building programme could be maintained and settlements during and after construction of the building were within tolerable limits. In addition, the drain installation and preloading periods allowed time for detailed structural and plant design and in effect assisted the building programme as the sanddrains could be installed much earlier than a piling system.

GROUND INVESTIGATION

The programme allowed about one month for a second stage site investigation, including laboratory tests, and design proposals for ground treatment. Two 0.2m diam. boreholes were constructed in which 0.102m diam. Geonor piston samples were recovered at close intervals. Some in-situ vane tests were also made with a Farnell Apparatus. Figure 2 gives a summary of the soil profile and the soil parameters. Based on field vane, laboratory vane and undrained triaxial compression tests, the mean cohesion is shown to increase from 11 kPa near surface to 18 kPa at 14m, giving an allowable increase in load of about 25 kPa for a raft foundation.

Several of the samples were extruded and allowed to air dry so that the soil macrofabric could be qualitatively assessed in conjunction with the laboratory consolidation test results (which were necessarily on oedometer specimens cut vertically due to the time constraints). This data showed the presence of two weak clay layers of low permeability - an upper layer from 1.9m to about 7.5m with a mean Cv of 7.9×10^{-8} m^2/s and a lower layer from 10m to 14.1m with a lower mean Cv of 4.1×10^{-8} m^2/s. These two clay layers are separated by a clayey sandy silt, the lower levels of which were considered as a drainage layer in a first analysis of normal vertical consolidation without pretreatment.

To determine the amount and rate of consolidation settlement, the soil below the proposed raft foundation was divided into 7 layers as shown on Fig.2 and mean values of m_v allocated to each layer. (Layer 7 is a relatively incompressible interbedded silt, sand and clay with gravel layers, from 24 to 36m below surface). Under the proposed average continuous load of 90 kPa from a raft foundation, a total settlement of about 0.45m was expected and the calculated rate of settlement in layers 2 - 5 without sanddrains was relatively slow.

STRUCTURE

The building houses plant in which pharmaceuticals are prepared. The 40m x 28m reinforced concrete raft is 1m thick and was placed on 0.075m of blinding which lies on granular fill comprising 0.5m of slag remaining from the preload embankment and 0.15m of rolled crushed rock. The steel frame structure is infilled with brickwork or cement blocks and many of the internal 0.125 to 0.2m thick concrete floors have superimposed impermeable screeds of between .060m and 0.19m thick laid to falls to facilitate maintenance. The internal walls also have special flexible fine finishes to inhibit crack formation in the sterile areas. The South elevation of the 26m high steel structure is shown on Fig.1. The eastern 6-storey side of the structure houses large items of plant. The western 4-storey section is of the order of 10% lighter and contains offices, some plant and miscellaneous facilities.

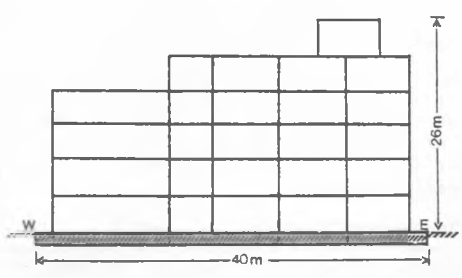

Fig.1. South elevation of steel frame

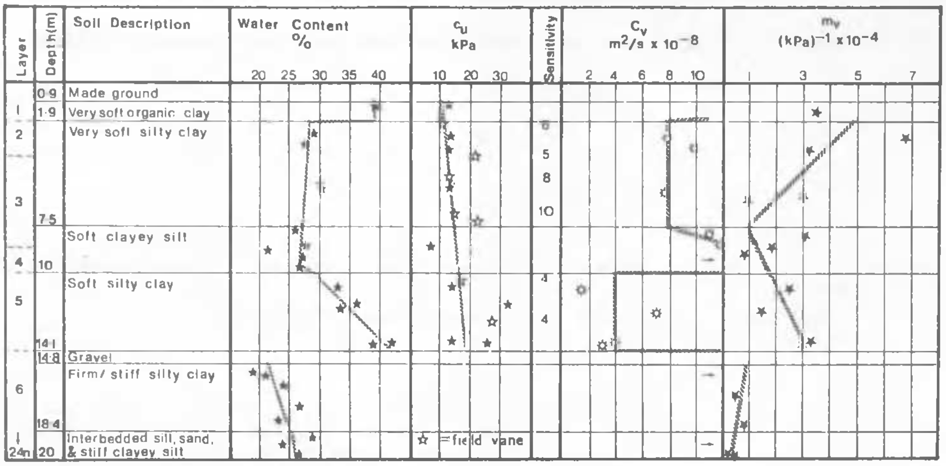

Fig.2. Soil profile

PRELOAD EMBANKMENT CONSTRUCTION

The design of the ground treatment spacings and the preloading programme was partly predetermined by a commencement date for the building works which was due some 8 months after the order to start sanddrain installation. To allow for possible future design and structural changes, a minimum preload of 107 kPa was specified initially. A surcharge of about 13% was effectively added by later adjustments. With a target of 90% consolidation settlement under full embankment load, the equivalent of at least 100% settlement under the maximum design load could be expected.

The first 0.6m of slag fill was placed on a sheet of Terram in July 1974 in order to provide a working surface for sanddrain installation. In early September 1974 the filling of the preload embankment commenced immediately after wick and instrumentation installation. The slag was brought by lorry from the fill source 300m distant, end tipped and graded level. A ramp was built onto the East side of the embankment for lorry access.

A consideration of the initial stability resulted in a permitted first increment of 2.8m which was actually placed over two weeks. The subsequent rate of filling of 0.75m per week was determined from an approximate curve fitting method which utilises parameters from the oedometer and consolidated undrained triaxial compression tests. (Wong 1971). The final total fill height of 6.5m was attained in a total of nearly 7 weeks, on 23rd October 1974, only a few days later than scheduled.

From in-situ density checks, the fill was proved to have an average bulk density of 1860 kg/m^3. The embankment top edge extended about 2m beyond the proposed concrete raft area on all sides and was constructed with side slopes of 1 to $1\frac{1}{2}$. Between 5 and 7m from the western toe of the preload embankment buried services were located which consisted of a 0.05m diameter water pipe, a cable duct and a 0.45m diameter sewer.

After completion of the preloading in March 1975, the embankment was removed by dozer to form a working surface in an adjoining development area some 30m from the East toe of the embankment. This re-utilisation of the slag material was another factor which allowed an economic preloading scheme.

SAND DRAIN SPACING DESIGN

The design of sanddrain spacings (s) has been developed by Barron (1948) and is based on the relationship

$$C_h \times t = 4R^2 \times T_h \qquad \text{..........(1)}$$

where C_h is the coefficient of consolidation with radial drainage, t the time to achieve a specified degree of consolidation, R the effective radius of influence of the drain of radius r (where 2R = 1.13s for a square drain grid) and T_h the time factor which is related to the ratio $n = R/r$.

The necessary embankment construction period and the overriding requirement to commence raft construction in March 1975 determined the available consolidation period (t) of 18 weeks. The normal allowance of half the embankment construction period was ignored in order to introduce a short "float" time element in the programme (equal to 3 weeks in the original programme time at the spacing design stage). This additional time element was also intended to make some allowance for variation in soil parameters across the site.

The adverse effects of smear and disturbance which might occur in forming sanddrains are not quantifiable although attempts have been made to provide a rational design approach. Hence, consolidation by vertical drainage was discounted in an attempt to offset these factors. Close inspection of the soil specimens suggested that the macrofabric had some horizontal features which would allow a factor in excess of unity when attempting to assess a C_h value from the laboratory C_v values. However, features such as silty laminae were at infrequent intervals, often subvertical, and were absent at many levels. (See McGown et al 1979). A value of C_h was chosen for equation (1) equal to the mean oedometer C_v value of 4.1×10^{-8} m^2/s in Layer 5.

Using a Sandwick diameter (2r) of 65mm a spacing (s) of 0.9m on a square grid was chosen, which, within the foregoing limitations, was designed to achieve 90% of the total consolidation settlement. The model predicted about 80% consolidation settlement in Layer 5 at the end of the 18 weeks consolidation period whilst in Layers 2 and 3, some 95% of the consolidation settlement was anticipated. The embankment load placed after Sandwick installation was approx. 110 kPa and during preloading the expected total settlement within Layers 1 to 7 below the embankment centre was 0.472m on the basis of one-dimensional consolidation tests. In addition, elastic settlements of 41mm were anticipated. The laboratory oedometer tests indicated that settlement within layers 1, 4, 6 and 7 would occur soon after imposition of any loads.

EMBANKMENT CONTROL

Curves of excess porewater pressure against embankment height were calculated for each piezometer. Elastic methods were used to derive the stress factor at the different piezometer levels and positions. Values of the porewater pressure parameter A ranging from 0.33 to 0.53 were obtained from laboratory consolidated undrained triaxial compression tests. A stability envelope was devised for each piezometer and actual piezometer readings during preloading were plotted as a curve of excess porewater pressure against embankment height. A safety factor of 0.7 was applied to the expected excess pwp for a total embankment height in excess of 3.4m. (See Brons and Alusi 1973).

CONTROL INSTRUMENTATION

A plan of the instrumentation layout is given on Fig.3. The instruments were installed after Sandwick installation. During embankment loading, a set of instrument readings was taken at approximate 0.4m increments of filling, i.e. twice per week. This procedure limited the sitework on readings to about 6 hours and enabled all the data to be evaluated and transmitted by telephone to the controlling engineer by the following morning. The frequency of readings on some instruments was varied during and after embankment construction in order to maintain a rapid analysis of the essential data from piezometers and heave pegs.

Pneumatic piezometers were installed in two main groups, one below the embankment centre and the other below the North top edge of the filled area. Two further piezometers were installed outside the treated area at depths of 3m and 6m respectively below original ground level beneath each of the embankment side slopes. The static ground water level was determined from a Casagrande piezometer some 50m away from the embankment and was used as a datum for the measurement of excess pore water pressure in all piezometers beneath the embankment.

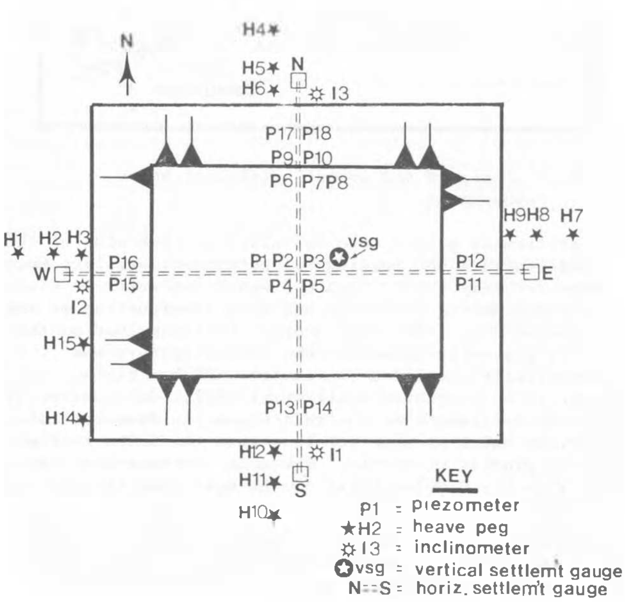

Fig.3. Instrumentation layout

SANDWICK INSTALLATION

The Sandwick installation unit comprises a tracked crane from which vertical leaders are suspended. A 0.095m od steel casing is driven into the ground with the assistance where necessary of a high frequency hydraulic vibrator. The casing has a disposable shoe on its base.

The Sandwick comprises a 0.065m diameter seamless woven polypropylene stocking which is pneumatically filled on site with washed and graded sharp sand. (See Hughes 1972). Some 1850 Sandwicks were installed with a single drilling unit to an average depth of 15m in $2\frac{1}{2}$ weeks in August 1974.

BEHAVIOUR UNDER PRELOADING

a) Piezometers

Figure 4 shows the response of piezometers P3, P4, P7, and P9 during preloading.

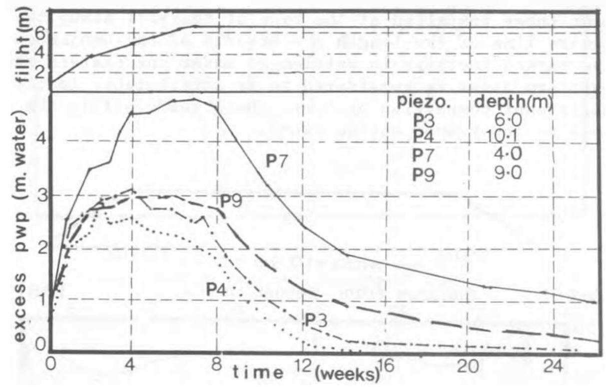

Fig.4. Piezometer excess porewater pressures during preloading

The excess porewater pressure in Piezometers P11 (at 3m), P12 (6m), P16 (6m) and P18 (6m) moved above the stability envelopes during preloading. However, the embankment geometry and density at the North and East sides was at variance with the initial stress increase assumptions due to the higher fill density caused by wheel loads on the access ramp. At P16, a cessation of loading was ordered until a detailed inspection revealed that the embankment top edge extended a short distance beyond that anticipated in the initial calculated response. The recalculated envelope for the revised fill geometry showed the excess pore pressure to be at an acceptable level.

The standing water level monitored in the Casagrande Piezometer varied over a small range - evidently in response to rainfall. Despite the proximity to the sea, no variations simulating tidal movements were detected.

Figure 5 shows the calculated degree of dissipation of excess porewater at the piezometer depths during the period after the embankment had reached full height. Evidently, in the soils to 10m below the embankment centre, an average 90% dissipation was achieved within 3 weeks of completion of filling whilst, between 10m and 14.5m, the extrapolated time for 80% consolidation is about 5 weeks. Below the North top edge of the embank-

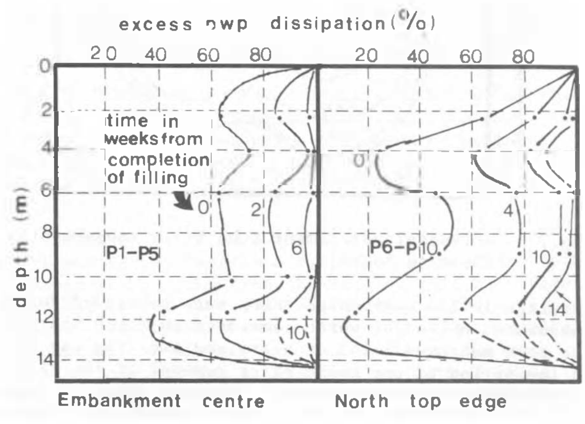

Fig.5. Contours of excess porewater pressure dissipation after completion of filling

ment the degree of excess porewater pressure dissipation in Piezometers P6 to P10 is generally less at all levels at any comparable time, and, although closer to the predicted times, the overall rate of consolidation was quicker than anticipated.

b) Settlement

Figures 6 and 7 show the movement of horizontal settlement tubes installed at the base of the fill along the centre line of the length and breadth of the embankment. The marked increase in settlement under the eastern and northern sides is considered to be attributable to the additional compaction by lorry wheel loads within the area of the ramped access route.

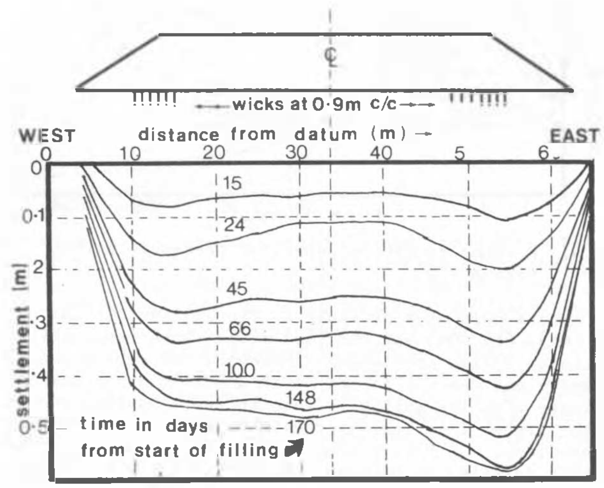

Fig.6. Horizontal settlement tube below embankment (West to East)

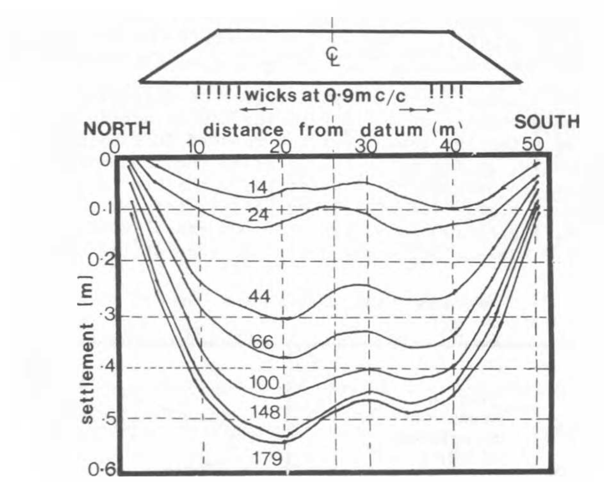

Fig.7. Horizontal settlement tube below embankment (North to South)

Readings in the horizontal tubes were continued during embankment unloading which commenced on March 6th 1975. The mean measured total of heave and swelling was 0.04m in the period of one month after removal of the fill commenced. The abstraction of the fill material forming the embankment occupied the first 2 weeks of this period. In comparison, a calculation based on cyclic swelling tests on the oedometer specimens gave a total of 0.076m movement. The horizontal settlement tubes were subsequently disturbed during preparation of the foundation layer of slag.

Figure 8 shows on a logarithmic scale the settlement against time of two points, one below the embankment centre and the other below the North top edge of the embankment. The predicted settlement curve by comparison slightly over-estimated the total movement below the embankment centre by 0.029m, i.e. a 6% difference. The shape of both the predicted and actual settlement/time curves is similar. It is evident that 90% and 100% settlements below the embankment centre occurred at about 10 weeks and 16 weeks from completion of filling. The graph also suggests that the total settlement derived from these settlement curves at any point in time lags behind the consolidation settlement inferred from the degree of excess porewater dissipation in the piezometers. However, the calculated maximum instantaneous porewater pressure response under full embankment load used for comparison purposes, is evidently excessive.

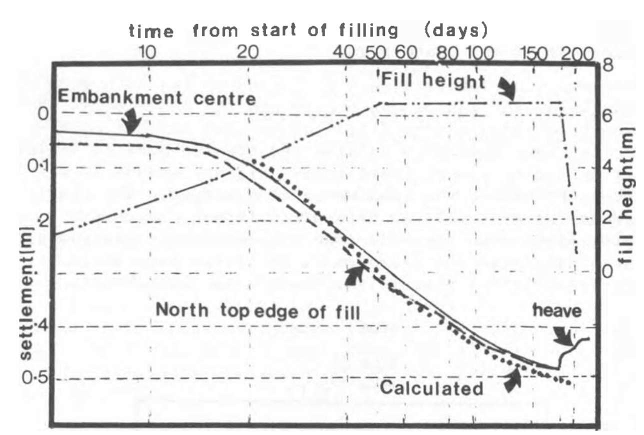

Fig.8. Calculated and actual settlement below embankment

BRE settlement points were installed on each side of the foundation raft and readings of settlement were recommenced on 1st July 1975. Figure 9 shows the settlement at each point during and after building construction in the 5 years to June 1980. The average building load on the soil is presently about 90 kPa. The structure was substantially completed in January 1977 but plant installation continued until April 1977. In January 1977 the mean settlement of the raft since its construction was 0.03m which is some 50% less than the 0.06m settlement originally predicted. The mean settlement on the East side was 0.045m whilst at the West side it was 0.012m.

The North East quadrant of the 6-storey section of the building is now the heaviest due to a concentration of plant and a preponderance of floor screeds and wall finishes.

In June 1980 the mean settlement on the raft was 0.055m compared with the calculated movement of 0.085m derived from a consideration of the probable elastic movements during building construction and secondary consolidation settlement.

The mean secondary consolidation coefficients for each layer which were used in the predictions were derived from oedometer tests and are given in Table I. From an analysis of the raft movements since April 1977 a mean coefficient of 0.0025 was obtained which is nearly 50%

lower than the weighted mean of the laboratory coefficients.

Table I

Layer	Coeff. of Secondary Consolidation
1-4	0.0051
5	0.010
6	0.0018

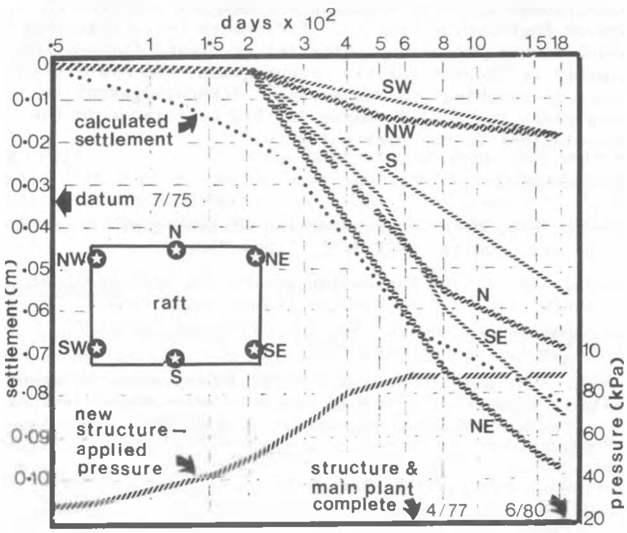

Fig.9. Settlement of raft during and after building construction

c) Heave pegs

Heave pegs were positioned at distances of 1m or 2m, 5m and 10m away from the embankment toe. These pegs were accurately levelled frequently during embankment construction in order to assist in allowing the onset of any instability to be monitored.

Negligible heave occurred at all positions but settlements at 1 or 2m lateral distance from the toe were in the range between 0.073m and 0.113m at completion of the preloading period. At 5m from the toe settlements were in the range from 0.025m to 0.041m whilst at 10m only small movements of between 0.005 and 0.013m were measured. These results confirmed a pre-contract calculation which was required to establish the minimum distance at which the toe of a possible second preload embankment could be placed at a later date so as not to affect, with any significance, the present building. Figure 10 shows the settlements at the heave pegs on the West and South sides of the embankment. The final movements at surface above the buried services beyond the West toe are of the order of 0.025m.

d) Inclinometers

Three inclinometer tubes I1, I2 and I3 were installed below the toe of the fill to a depth of 16 metres. Displacements from the original vertical tube position were evaluated by use of a computer programme. The tube at the west toe of the embankment was again of importance in forecasting any adverse movement near the service pipes whilst to the North side of the fill, a concrete access road had to be maintained in good condition. A maximum lateral movement of 64mm occurred at 4m below the embankment toe at 2.7m below natural ground level in Tube I1 on the South side. In I2 and I3

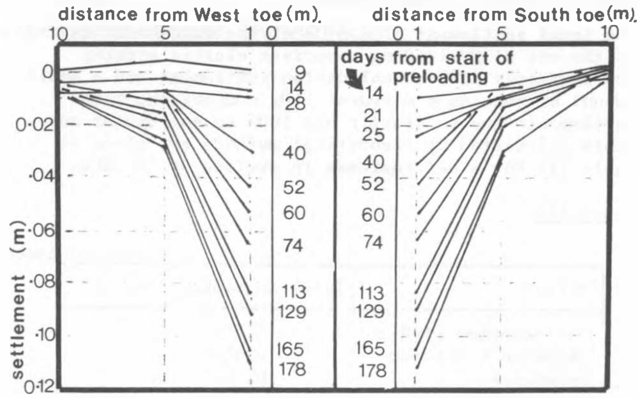

Fig. 10. Settlement beyond embankment toe

maximum lateral movements of 50mm occurred at 2.5 to 3m below embankment toe level. The maximum lateral movements below 9m from surface are all less than 0.025m.

BACK ANALYSIS

An analysis of the performance of the piezometers in the treated area during preloading was made using a curve-fitting method. (Richart 1959). The resultant field C_h values are shown on Table II for two positions below the embankment.

Table II

Depth (m)	Field C_h $m^2/s \times 10^{-8}$	Lab. C_v $m^2/s \times 10^{-8}$	Ratio $\frac{\text{Field } C_h}{\text{Lab. } C_v}$
Embankment Centre			
2.1	20.6)	mean	
4.0	21.2)	=7.9	2.5
6.0	18.1)		
10.1	20.6	Rapid	1.0
11.5	12.4	4.1	3.0
Embankment North Top Edge			
2.3	18.4)	mean	
4.0	8.9)	=7.9	1.7
6.0	13.3)		
9.0	13.0	15.9	0.82
11.5	9.5	4.1	2.3

Except in the variable layer between 7.5m and 10m the ratios of field C_h to laboratory C_v are in a small range of 1.7 to 3.0. Evidently the clay content in Layers 3 & 4 (See Fig.2) is variable over the site, thus significantly affecting permeability at isolated piezometer levels.

Mean values of C_h of 10.5×10^{-8} m^2/s and 9.8×10^{-8} m^2/s respectively have been derived for the full depth of compressible soils from the vertical and horizontal gauge readings at the embankment centre. These results are about two-thirds the mean field C_h value from the back analysis on piezometers and, although the more accurate, they are again dependent on curve fitting methods. Small amounts of secondary compression may well be included within the movements used in the calculations. By comparison, the weighted mean for field C_h derived from piezometers is 16×10^{-8} m^2/s.

The total settlement of 0.479m at the embankment centre at the end of preloading comprises elastic strain, approximately 100% consolidation settlement and a small amount of secondary movement. By comparison, total settlements due to elastic and 100% consolidation movements calculated by theoretical methods are given in Table III for a net increase in preload of 107 kPa.

Table III

Method	Total Settlement (m)
Steinbrenner (1934)	0.474
Skempton & Bjerrum	0.408
Oedometer	0.513

The elastic settlement included in the Table III is 0.041m and was derived from assumed values of Youngs Modulus E_n. Using the stress/strain curves from the consolidated undrained triaxial compression tests, lower E_n values of the order of 6000 kPa were obtained at 2% strain at sample depths of 2m and 5m. At 11m, an E_n value of 8800 kPa was obtained at 2% strain. With these two E_n values, the calculated immediate settlement would be 0.067m which would increase the total settlements in Table 3 by about 0.026m.

CONCLUSION

The use of Sandwicks allowed a safe preloading and a relatively high steel frame structure to be placed on poor compressible soils within the required programme time and without subsequent intolerable settlements to the structure. With the hindsight of the actual consolidation parameters, the sanddrain spacings could have been slightly wider with a consequent small financial saving. However, the structure benefited from a preloading to about 100% consolidation settlement at some 10% above probable maximum future design load and reduced the expected movements during and after building construction.

In the particular circumstances of this case history, the additional cost of and the resultant delays caused by an extended soils investigation would have absorbed much of any saving. The use of a piled foundation would have cost almost double the cost of Sandwicks and preloading. Also, a piled foundation would have incurred a delay in the building programme as the structural design could not be finalised until late into the preloading period.

The prediction of field C_h values was made safely by the use of Oedometer results but the back analyses showed that the ratio of the field C_h (from both piezometer and settlement results) and laboratory C_v varied from below unity to 3 on a relatively small site which has a comparative degree of uniformity of soil conditions. (See Garassino, Jamiolkowski et al 1979).

The use of a displacement method of installing wick drains did not cause a measurable additional settlement due to the disturbance and remoulding. The amount of settlement during preloading was within 10% of the prediction using data from Oedometer tests on high quality samples. The mean secondary movement to date is below the amount anticipated although the comparatively uneven distribution of load over the raft foundation has caused a relatively wide variation in movement between the East and West ends of the raft. The total differential movement between the raft centre (obtained from the mean of the movements at the North and South BRE settlement points) and the NE settlement point is of the order of 1 in 660 which is within the acceptable tolerance. Moreover, a proportion of the movements occurred during erection of the structure so that the brick and block walls will have experienced less differential movements.

ACKNOWLEDGEMENTS

The authors express their thanks to Glaxo Operations (UK) Ltd., for permission to publish the information contained within this paper. The assistance of the Chief Engineer, K.E.Brown, Esq., and his staff at Ulverston is gratefully acknowledged. The paper is published with the permission of J.L.Carlile Esq., Managing Director of Cementation Ground Engineering Limited. F.H.Hughes Esq., Director, Geotechnical Department, Cementation Ground Engineering Limited is thanked for his encouragement and his assistance in checking the manuscript. Acknowledgement is also given to Mr. J.M. Bayne for his assistance in the calculations in the back analysis.

REFERENCES

Barron, R.A. (1947) Consolidation of Fine-grained soils by drain wells. ASCE 113. 718 - 742.

Bayne, J.M. (1978) Foundation preloading with Vertical Sandwicks - a Case History (Unpublished report submitted as part of the MSc course at Imperial College, London University.

Brons, K.F. and Al Alusi A.F. (1973) Comparison between soil parameters obtained in the laboratory and values calculated from an instrumented surcharge. Proc.8th ICSMFE (3) 19 - 24, Moscow.

Dastidar, A.G., Gupta, S., Ghosh, T.K. (1969) Application of sandwicks in a housing project. Proc.7th ICSMFE Mexico, V2. 59 - 64.

Garassino, A., Jamiolkowski, M., Lancellotta, R. and Tonghini, M. (1979). Behaviour of pre-loading embankments on different vertical drains with reference to soil consolidation characteristics. Proc.7th ECSMFF (V3) 213 - 218, Brighton.

Hughes, F.H., Chalmers, A. (1972). Small diameter sand drains. Civ. Eng. & Pub. Wks. Review V.67 No. 788.

McGown, A., McNeill, N., Gabr, A.W. (1979). Predicted and measured behaviour of Clyde alluvia beneath Renfrew Motorway Stage 2 embankments. Proc.7th Europ. Conf. Soil Mech. Found. Engg. (V3) 231 - 237 Brighton.

Richart, F.E. Jnr. (1959). A review of the theories for sanddrains. ASCE V.124 709.

Wong, H.Y. (1972). General design procedures for sand drain foundations. (Unpublished). Cementation Research Ltd.