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Two Road Embankments on Soft Alluvium

Deux Remblais Routiers sur Argile Molle

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SYNOPSIS Two new by-pass roads under construction in Kent cross extensive areas of soft alluvium on embankments rising up to 8 metres above the original marsh surface. Initial design following site investigation over-estimated the drainage characteristics of the alluvium with the result that neither embankment could be completed as planned. An unusual feature of the soft alluvium is the very low value of the coefficient of consolidation when the vertical effective stress exceeds the preconsolidation pressure. After extensive site trials, during which a recently developed method of determining the in situ drainage characteristics of soils at stresses well in excess of existing stresses was used, additional drainage was installed in the alluvium and the embankments completed. The results obtained from the in situ tests and the observed embankment behaviour have been analysed using a new computerised method and have been shown to be in good agreement.

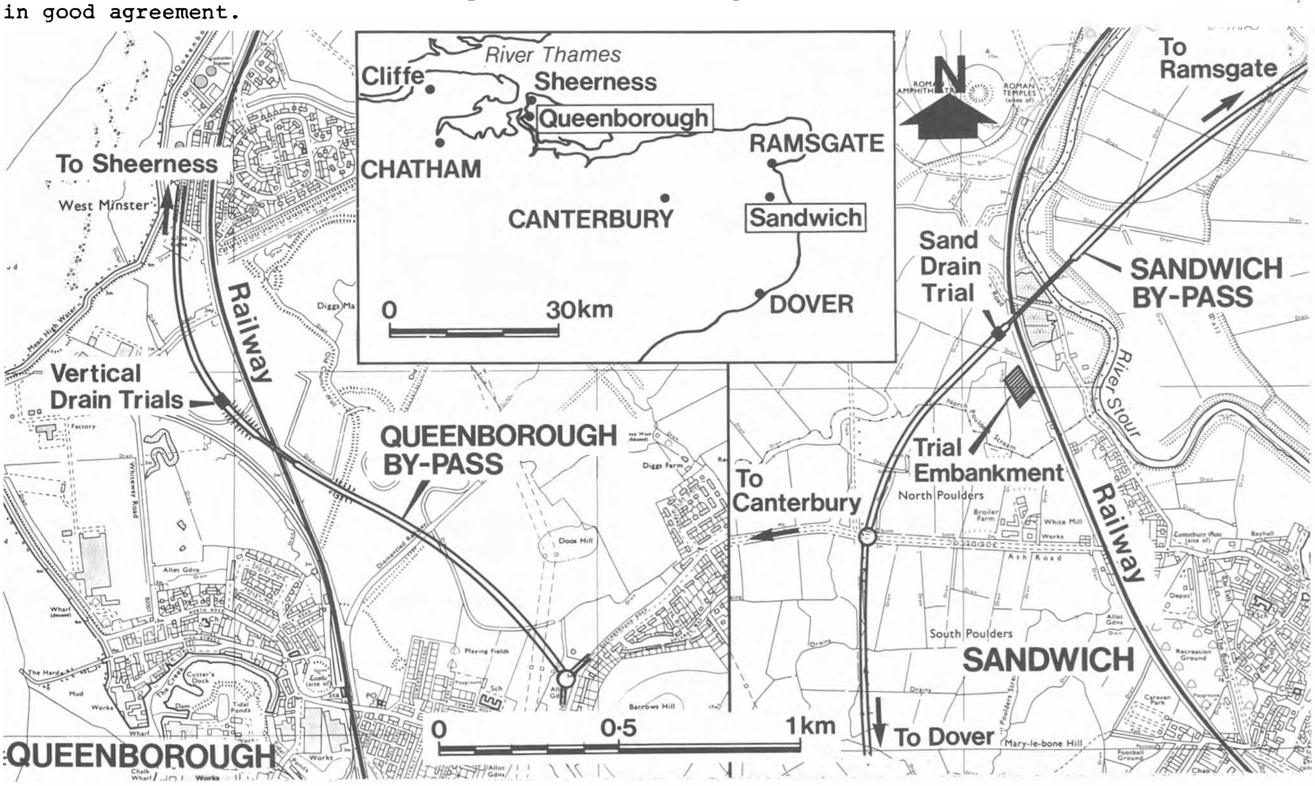

Fig. 1 Site locations and site plans

INTRODUCTION

At Sandwich in East Kent and Queenborough on the Isle of Sheppey in North Kent, new by-pass roads to the towns are being constructed (Fig. 1). The alignments of these new roads require them to pass over existing railway tracks which cross extensive areas of soft alluvium, marsh, on low embankments. This results in the road surfaces having to rise between 6.5 and 8 metres above the original surface of the marsh on embankments and viaducts. The predicted total settlements of the highest parts of the embankments adjacent to the viaducts over the railways are between 1.2 and 1.5 metres.

At Sandwich an instrumented trial embankment was constructed in advance of the main construction whereas at Queenborough the first stage of the

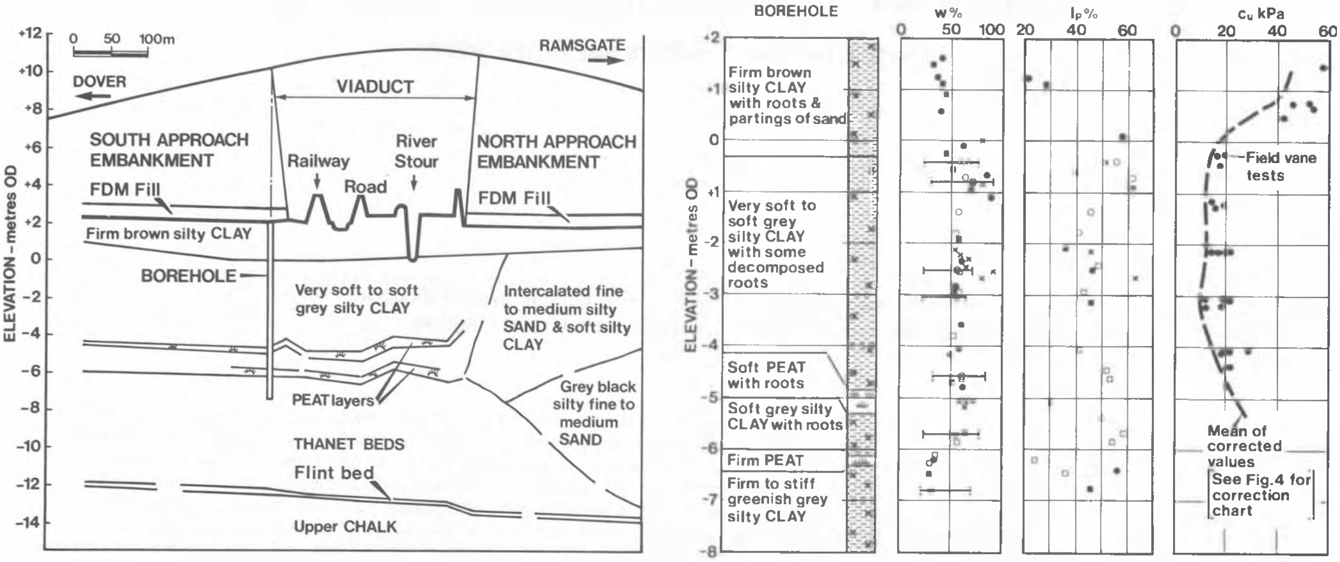

Fig. 2 Sandwich By-Pass. Longitudinal section, borehole log and soil properties

road embankment was instrumented. The initial results from these trial embankments and other published case histories were interpreted as showing that the road embankments could probably be built without additional drainage being installed. However, at both sites as construction of the road embankments progressed it became apparent that they could not be completed by the required dates. At this stage KCC requested Ove Arup and Partners to give advice on alternative ways of completing the embankments and at both sites it was concluded that additional drainage of the underlying alluvial clay, together with small extensions of the lengths of the viaducts and slight lowering of the finished road levels, was the most appropriate and economic solution. Trials of alternative proprietary drainage systems were carried out at both sites from which the final design of the ground drainage schemes and the embankment construction programme were determined. Construction of both embankments is now nearing completion enabling the measured performance of each road embankment to be compared with that predicted.

GROUND CONDITIONS

The ground conditions at the two sites are similar, consisting of alluvial deposits of Pleistocene to recent age and approximately 10 metres thick, deposited above an erosion surface near the confluence of river channels and the sea. The existing surfaces of the marshes are 1 to 2m above mean sea level and, although they are protected by flood banks, at times of heavy rainfall these areas become flooded irrespective of the sea level. The marshes have been reclaimed in stages for agricultural use by the construction of low embankments and drainage ditches across them, the embankments being constructed from the adjacent near-surface materials. The alluvium thus has an irregular surface and a somewhat complex stress-history resulting particularly in varying degrees of overconsolidation.

Sandwich

The soil profile and properties in the vicinity of the highest part of the road embankments at Sandwich are shown on Fig. 2. South of the

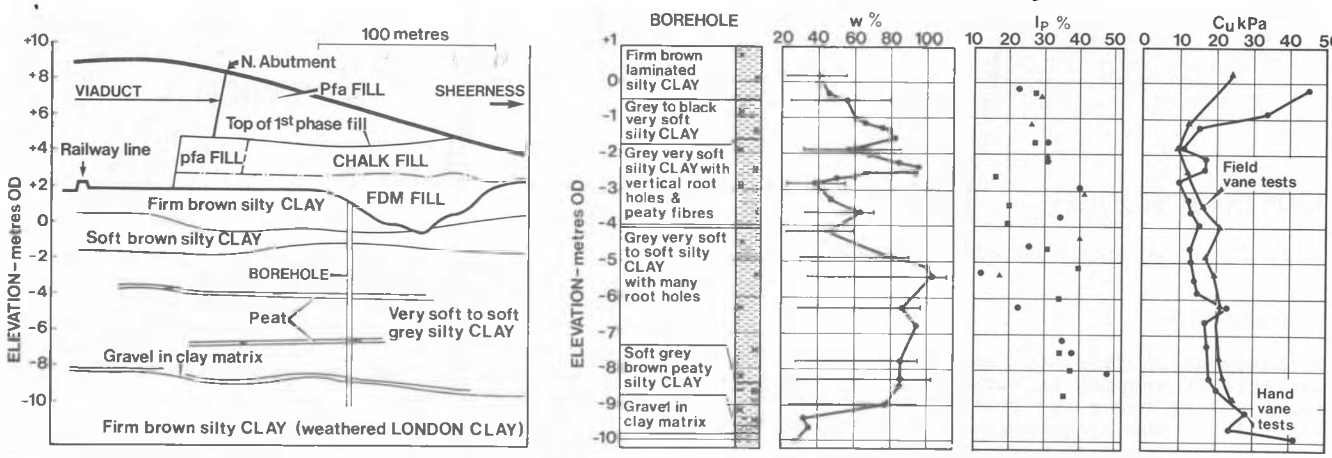

Fig. 3 Queenborough By-Pass. Longitudinal section, borehole log and soil properties.

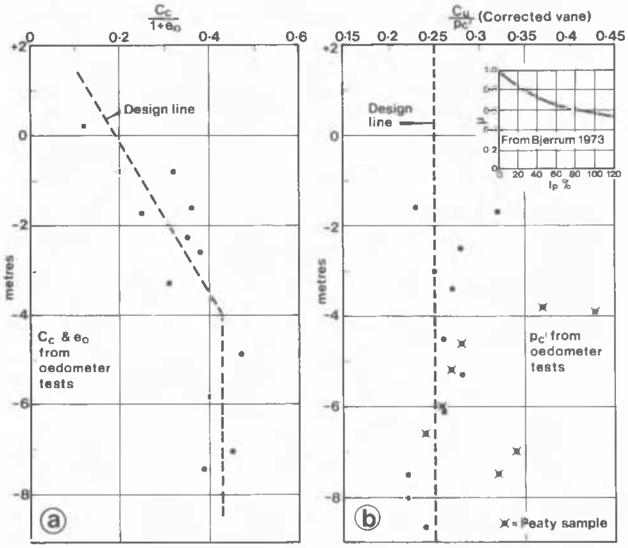

Fig. 4 Queenborough By-Pass
a) Compression ratio, and b) c_u/p_c' with depth

River Stour the alluvium is approximately 8 metres deep overlying the eroded surface of the Thanet Beds of Palaeocene Age. The Thanet Beds comprise firm to stiff silty clay approximately 6 metres thick which in turn overlie the eroded surface of the chalk of Cretaceous Age. The alluvium here consists of three layers; a firm desiccated crust of silty clay approximately 2 metres thick overlying 3 metres of soft to very soft lightly over-consolidated silty clay which in turn overlies 3 metres of soft normally consolidated silty clay. Roots and root remains are prevalent in the desiccated crust whilst peat and peaty layers are present within the bottom two metres of the alluvium. The shear strength of the desiccated crust is typically of the order of 40 kPa. In the middle of the alluvium the shear strength is 10 to 15 kPa and at the base it is slightly higher at 10 to 30 kPa, but this apparent increase in strength could be due to the presence of the peat.

To the north of the River Stour the character of the alluvium changes to silty sands intercalated with thin layers of soft, silty clay, and the relatively high rate of consolidation permitted rapid construction of the north approach embankment.

<u>Queenborough</u>

The soil profile and properties in the vicinity of the highest part of the road embankments at Queenborough are summarised on Fig. 3. The alluvium is approximately 11 metres deep overlying the eroded and weathered upper surface of the London Clay of Eocene Age, and consists of two layers; a firm desiccated crust of silty clay approximately 2 metres thick overlying very soft lightly overconsolidated alluvial clay. Within the desiccated crust and towards the base of the deposit roots and root holes are present. In addition, peat and peaty layers are present in the lower half of the alluvium. Throughout its depth the alluvial clay is often laminated and more permeable layers have been identified, including a layer of coarse gravel at the base of the alluvium immediately above the London Clay, but these appear to be of limited lateral extent. The shear strength of the desiccated crust at this site is approximately 30 kPa and the alluvial clay immediately below it has a shear strength of approximately 10 kPa increasing almost linearly to approximately 20 kPa at the base of the alluvium.

As shown in Fig. 4a the Compression Ratio, $\frac{C_c}{1 + e_o}$, of the alluvium increases with depth to about 6 metres reflecting the over-consolidation of the upper alluvium. Below 6m it remains constant to the base of the alluvium.

The ratio of the vane shear strength corrected in the manner described by Bjerrum (1973) to the pre-consolidation pressure (see Fig. 9b) is shown on Fig. 4b. For design purposes a uniform value of 0.25 was adopted. A similar relationship was obtained from the aniso-tropically consolidated triaxial tests. For the range of plasticity index of the soils tested, the increase in ratio to allow for plane strain field conditions is offset by the reduction that would have to be made for orientation (Ladd and Foott, 1974).

Similar results to those above were obtained for the derived soil design values for the Sandwich site.

SANDWICH TRIAL EMBANKMENT

The report on the site investigation concluded that measures to improve natural drainage would be required and that a trial embankment should be constructed to obtain detailed information about settlement and stability.

Sandwick drains (Ground Engineering, 1978) made of 65mm diameter porous fabric stockings filled with medium sand were installed in wash-boreholes on a square grid at 1.15m centres under part of the trial embankment, see Fig. 5a. The trial embankment was completed within a 26 day period during September 1973 to a height of 3.08m above marsh level (Fig. 5b), which was the assessed maximum height which could be built quickly. The embankment fill generally comprised colliery spoil (unit weight approx-imately 21 kN/m^3) over a 0.6 metre thick layer of free draining material (F.D.M). The embankment remained at the 3m height for approximately 650 days, during which time approximately 550mm of settlement occurred where sand drains had been installed as compared to approximately 350mm in the untreated alluvium (Fig. 5e).

Fig. 5c shows the distribution of piezometric head with depth at various times after the completion of the filling. Initially the piezometric heads in the drained and undrained areas were similar, but the rate of dissipation in the area without sand drains was significantly slower than that of the area with sand drains. After 650 days approximately 90% dissipation of the excess pore pressure within the drained area

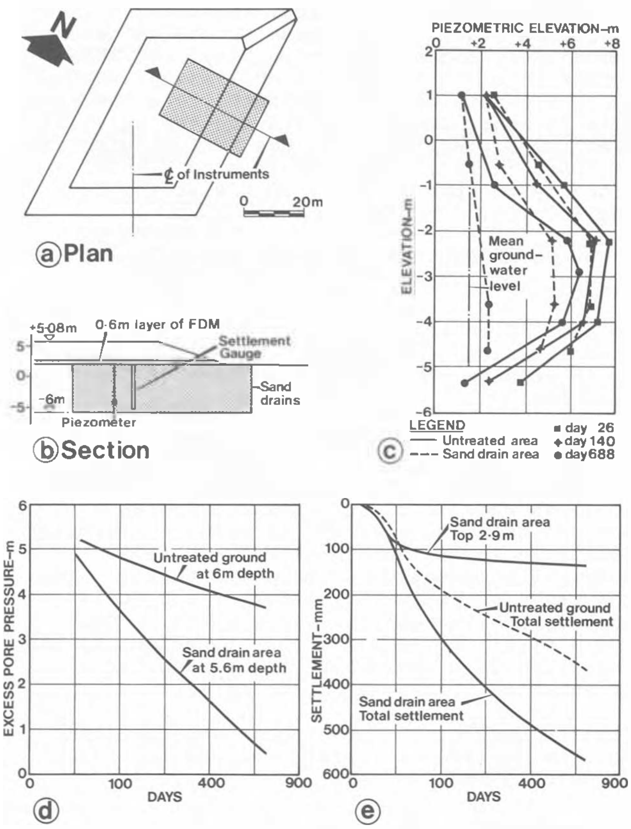

Fig. 5 Sandwich By-Pass Trial Embankment
a) Layout, b) Section, c) to e) Results

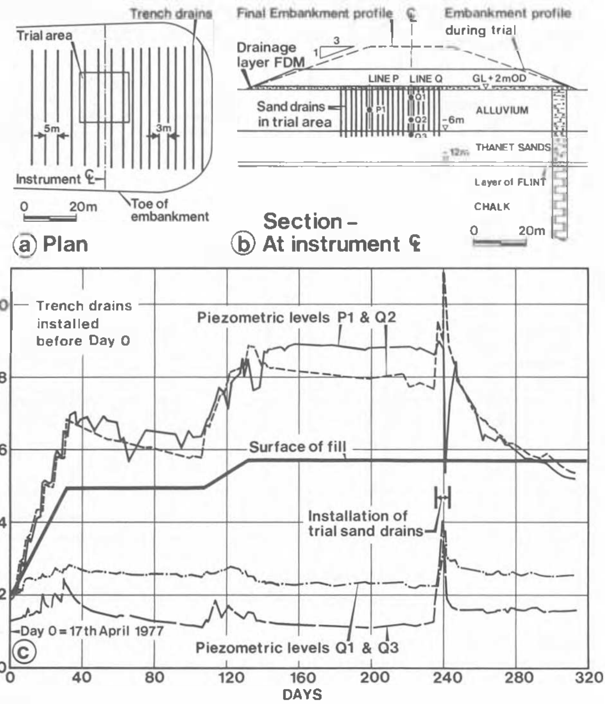

Fig. 6 Sandwich By-Pass. a) Position of Trench Drains and Sand Drain Trial, b) Section, c) Results of measurements.

had occurred compared with 30% in the untreated alluvium (Fig. 5d). Interpretation of the results of the trial embankment was made initially on the basis of the first 144 days of observations, which, it transpired was too short a period to properly discern the relative rates of dissipation appropriate to the full construction period.
In the overconsolidated crust, rapid consolidation occurred (Figs. 5c and 5e), and this feature was maintained at higher effective stresses during construction of the road embankment, reflecting the lower plasticity index (Fig. 2) and overconsolidation to an average value of 60 kPa.

SANDWICH BY-PASS ROAD EMBANKMENT

Trench Drains

The drainage system initially installed beneath the road embankments consisted of 225mm wide vertical trench drains located as shown in Fig. 6a, excavated through the alluvium by a Steenbergen B-600 machine (Ground Engineering, 1977). These drains extended to approximately 5.5m below the surface of the marsh (Fig. 7a) and were backfilled with granular material. The 600mm deep layer of free-draining material was used as a working platform for the trenching machine. The drainage system was designed principally to ensure that the required degree of settlement was achieved within the construction period.

Construction to Day 230

Construction of the south approach embankment started in April 1977, using colliery spoil above the drainage layer and a height of 3m above original marsh level was reached after 30 days (Fig. 6c). After a delay of 80 days a further 0.8m of fill was added. The rate of dissipation of excess pore pressure in the alluvium was found to be similar to the untreated area beneath the Trial Embankment, and was significantly slower than that required. Lateral movement in the alluvium at the toe of the embankment slope was approaching 100mm (Fig. 14).

Collapse of the weak alluvial clay into the drainage trenches below about 3m depth is thought to have rendered them ineffective for dissipating the maximum excess pore water pressures which occurred between 4 and 6m depth. Piezometers installed at 4.5m depth showed little difference in piezometric head in the alluvium close to and midway between the trenches (Fig. 7a).

Sand Drain Trial

As rapid changes in the consolidation behaviour of the alluvium had been indicated in the vicinity of the river it was thought prudent to carry out a further sand drain trial. The trial area, 19m square in plan (Fig. 6a) was located within the south approach embankment and close to the intended position of the viaduct south abutment, where instrumentation already existed and where excess pore pressure dissipation had been particularly slow. Sandwick drains were installed at 1m centres through the alluvium

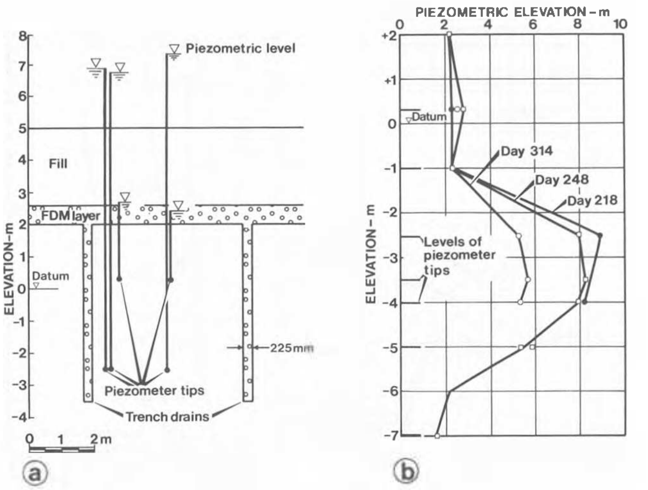

Fig. 7 Sandwich By-Pass. a) Piezometer readings at Trench Drains, b) Results of Sand Drain Trial.

from the surface of the 3.8m high road embankment (Fig. 6b).

The installation of the sand drains by driven mandrel caused a 3m increase in excess pore pressure, but this did not result in any detectable change in the stability of the embankment. It is thought that the increase in pore pressure was the result of a change in total stress, the effective stress remaining unchanged. After the initial rapid rate of dissipation of excess head to the value prior to the sand drain installation the rate of dissipation was confirmed to be similar to that measured at the Trial Embankment.

Completion of Embankment

Using the information from the Trial Embankment and Sand Drain Trial a layout of Sandwick drains was designed to permit stage construction of the remainder of the embankment using colliery spoil while maintaining a minimum factor of safety of 1.2. The rate of consolidation along the length of the approach embankment also had to be such that the road profile would be acceptable.

The spacing of drains varies between 0.8m centres at the viaduct abutment to 4.5m centres approximately 400 metres away where the embankment is about 4.5m high. The drains were placed on a square grid for the full width of the embankment including the side slopes (Ground Engineering, 1978).

Embankment filling recommenced on Day 382 (1st May 1978) and continued in stages until completed during April 1980. The progress of filling is summarised in Table 1.

QUEENBOROUGH BY-PASS ROAD EMBANKMENT

Construction to Day 700

The first stage of the road embankment was constructed as an instrumented trial embankment during the summer of 1976 and consisted of chalk fill with a unit weight of 18 kN/m^3 over a blanket of free draining material (Fig. 8b and 8c). Where a height of 3m was exceeded, substantial ground movements occurred and these sections were stabilised by reducing their height to approximately 3m.

Only some 20% of the dissipation of the excess pore water pressure from this first stage occurred during the 600 days after completion of the first stage of the filling (Fig. 8c). The pore pressure response in the alluvial clay beneath the embankment is summarised on Fig. 9b. The increase in pore pressure was approximately 5 kPa less than the increase in embankment loading. As shown on Fig. 9a, below the desiccated crust this difference is consistent with the difference between the vertical effective stress and the pre-consolidation pressure.

Some 50mm of lateral movement occurred at the toe of the 1 in 2 side slopes during filling and reached 70mm during the subsequent 600 days (Fig. 14).

Thus the embankments could not be completed, and particularly the required 95% consolidation in the vicinity of the piled viaduct abutments could not be reached by the required date of August 1981.

Vertical Drain Trials

Two different types of vertical drains were tested in the trial. These were the A.V. Colbond drains which consist of a non-woven polyester fabric strip 300mm wide by 4mm thick (Trial Area II) and Sandwicks (Trial Area I) see Fig. 8a.

Each Trial Area was 15m by 24m in plan with the drains extending the full depth of the alluvium (Fig. 8b). The Sandwicks of effective diameter 65mm were spaced at 1m centres, based upon the results obtained at Sandwich. It

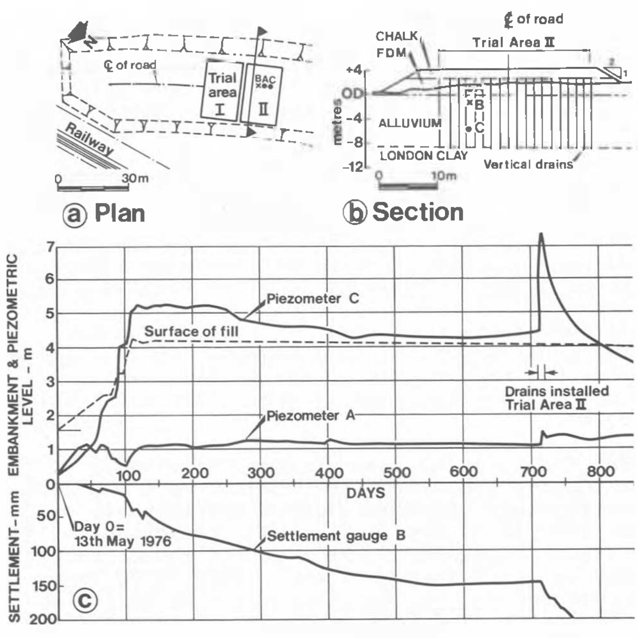

Fig. 8 Queenborough By-Pass. a) Vertical Drain Trial layout, b) Section, c) Results

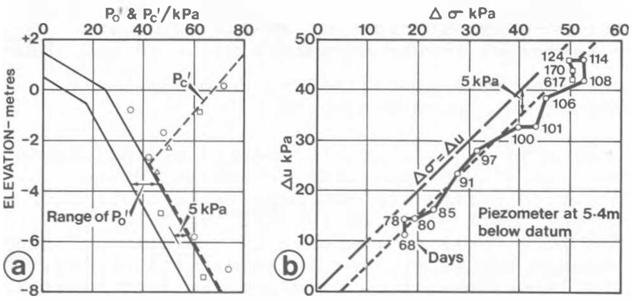

Fig. 9 Queenborough By-Pass. a) Vertical effective stress v. preconsolidation pressure, b) Change in piezometric pressure v. embankment pressure.

was assessed that the effective diameter of the A.V. Colbond drains was 220mm and that a spacing of 1.35m would be required to obtain the same rate of consolidation as the Sandwicks. As at Sandwich, predrilling through the embankment was necessary in order to install the vertical drains.

Installation of the drains was carried out during April, 1978, and as shown on Fig. 8c there was again an immediate 3m increase in piezometric head followed by rapid dissipation.

Comparison of Drain Performance

The change in piezometric head and settlement with time at various depths within the Trial Area I is summarised on Figs. 10a and 10b; a similar response was observed in Trial Area II. The pore water pressure response and settlement within the desiccated crust were small compared to that in the soft alluvium below.

As shown in Fig. 11a the dissipation of excess pore water pressures occurred more rapidly at all locations within the Trial Area I, the average percentage dissipation being 45% as compared to 32% with the Trial Area II during the first 90 days. During this same period the settlement of the Trial Area I was about 130mm compared to approximately 100mm in Trial Area II.

It was concluded that the effective diameter of the A.V. Colbond drains was 120mm and that in order to give comparable performance to Sandwick drains installed at 1m centres the spacing of the A.V. Colbond drains would have to be 1.15m. Somewhat larger effective diameters of band drains based on the circumference have been reported by Hansbo and Torstensson 1977 and Hansbo 1979. It is possible that the strip was folded inwards during placing, reducing the actual width below 300mm.

For the first 50 days dissipation closely matched a theoretical rate based on a coefficient of consolidation (horizontal) $c_{v(h)}$ of $1m^2/yr$. (Fig. 11a). The reduction in rate of dissipation evident from about the 50th day is consistent with the change in $c_{v(h)}$ indicated on Fig. 12. On this figure the results derived from oedometer tests and field permeability tests are plotted together with data published by Rowe 1972 and results from a nearby site at Cliffe (Fig. 1).

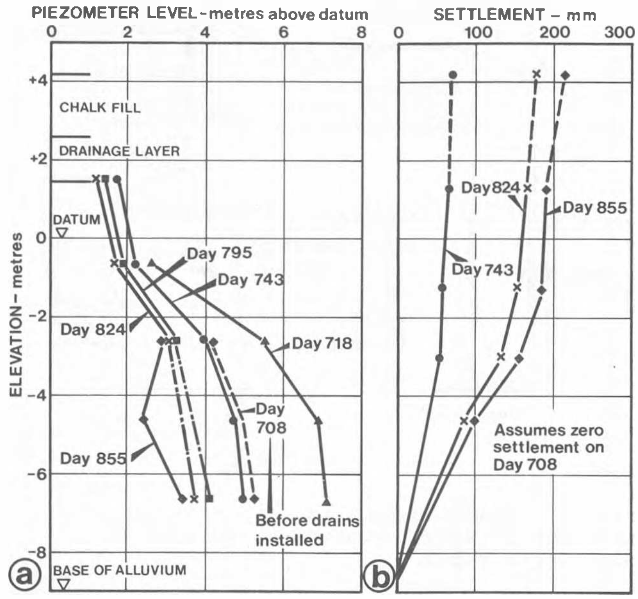

Fig. 10 Queenborough By-Pass. Changes in a) piezometric profile and b) settlement with time

At stresses only a small amount in excess of the preconsolidation pressure the values of $c_{v(v)}$ and $c_{v(h)}$ were found to average $0.5m^2/yr$. and converge on values between 0.2 and $0.4m^2/yr$. (Fig. 12). These unusually low values have been subsequently shown to be correct by analysis of the embankment behaviour and recent field tests (Fig. 12).

Rate of Consolidation

In the design of the drainage system a coefficient of consolidation of $0.7m^2/yr$. was used for vertical effective stresses up to 130 kPa and $0.35m^2/yr$. for vertical effective

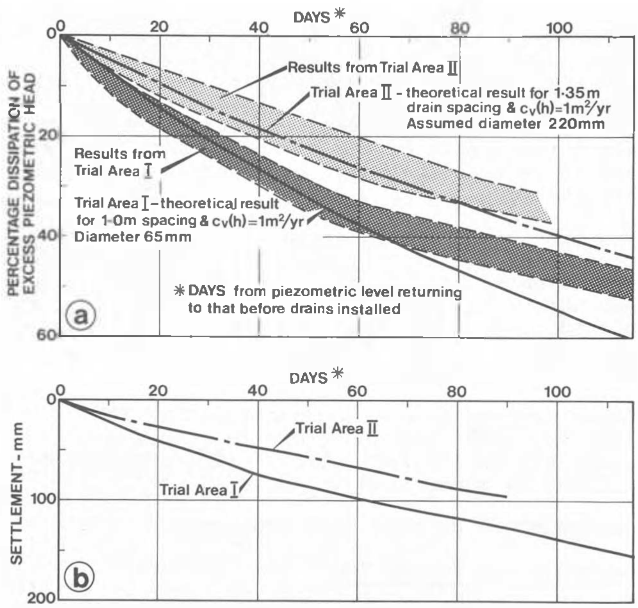

Fig. 11 Queenborough By-Pass. Vertical Drain Trials. Changes in a) dissipation and b) settlement with time.

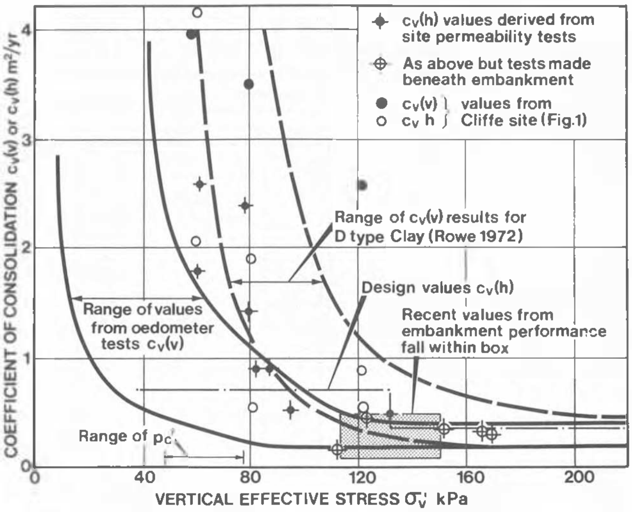

Fig. 12 Queenborough By-Pass. Variation in coefficient of consolidation with vertical effective stress.

stresses greater than 130 kPa (Fig. 12). Manual calculation of stage construction rates for a wide range of alternatives is tedious with more than one value of c_v. A computerised method of solution of permissible construction rate with reference to the current effective stress and continuously changing values of c_v has been developed (Cole 1980). This has been used in the back analyses of current values of $c_{v(h)}$ (Fig. 12).

Completion of the Embankment

Vertical drains were installed through the existing embankment and side slopes, and also beneath the areas upon which berms were constructed to enable completion of the higher parts of the embankment (Fig. 14). Sandwick drains were selected and the spacing was designed to allow stage construction of the embankment using pulverised fuel ash (pfa) with a unit weight of 13 kN/m^3 while maintaining a minimum factory of safety of 1.25. The spacing was also designed so that the required 95% primary consolidation was achieved at the piled abutment and settlement of the road was within acceptable limits. The drain spacing varied between 0.9 and 3.0m centres on a square grid.

Embankment filling restarted on Day 1063 (31st March 1979) and will be completed during 1980.

PREDICTED BEHAVIOUR AND MEASURED PERFORMANCE

Embankment construction at both sites was controlled by the use of charts giving limiting values of excess pore pressure related to embankment surface level (Fig. 15). When the excess pore pressure for a particular area of the embankment had fallen sufficiently, additional fill was added as shown in Fig. 15 for one area of the Queenborough By-Pass.

The relationships between the predicted settlement and the increase in loading due to the embankment construction for the two sites are shown on Fig. 13. In both cases the

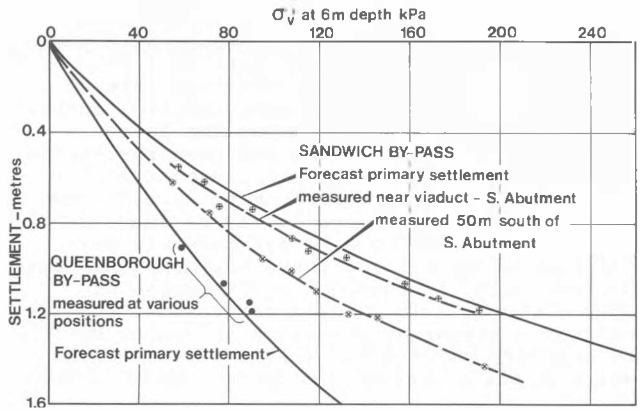

Fig. 13 Both sites. Settlement

predicted settlement based on the coefficient of compressibility obtained from conventional oedometer tests, was in reasonable agreement with the measured behaviour on site. There has been somewhat greater settlement than forecast in the area 50m south of the viaduct abutment at Sandwich (Fig. 13), and this may be attributable to disturbance of the ground during early trials of the trench drain equipment.

Settlement of the embankments has been accompanied by significant lateral ground movements (Fig. 14). The maximum lateral deformation which has occurred to date at both sites is approximately 200mm and this has been mainly due to strains within the alluvium. After installation of the vertical drains the average maximum horizontal movements were approximately 10% of the maximum settlements at embankment centrelines. During the construction of the Trial Embankment and during early stages of the road embankments before the vertical drains were installed the horizontal movement was much greater, being nearly 40% of the vertical movement.

The vertical drains were placed at the same spacing beneath the whole widths of the road embankments to ensure that the embankment structure would not be affected by wide disparities in settlement. This arrangement of vertical drains may have also have minimised the lateral migration of excess pore water pressures into the adjacent untreated ground and resulted in the smaller lateral movement after the drains were installed.

In Table I the predicted construction progress

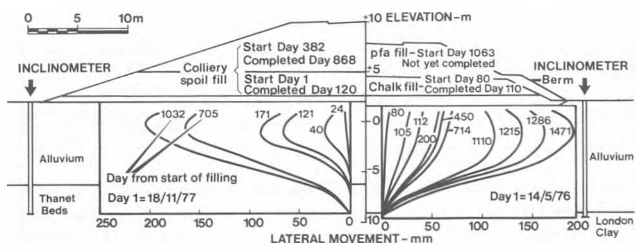

Fig. 14 Both sites. Embankment cross section and lateral movememnts.

after filling restarted is compared with that achieved. At Sandwich the rate of filling was always somewhat slower than the fastest rate permitted by the stability control charts, whereas at Queenborough during the latest stages the rate of filling has been restricted because the highest measured piezometric level has come close to the maximum permitted (*in Fig. 15). Nevertheless as may be seen from the estimates of the earliest possible day for placing each layer there has been only some 30 days delay at Sandwich and no delay at Queenborough. The reason for this is the low effective stress at which the values of c_v become very small (Fig. 12), a factor which was not fully appreciated at the design stage.

The initial design satisfactorily predicted the height of embankment which could be built immediately but the subsequent problems resulted from the difficulty in determining the appropriate value of the coefficient of consolidation relevant to the programme for the remainder of the embankment construction. Guidance from case histories published shortly before the initial design (Rowe 1968, 1972, Murray and Symons 1974), led to optimistic assessments of the likely field values of the coefficients of consolidation.

Kent County Council Highways Laboratory have developed an apparatus which enables accurate in situ determination of the coefficient of consolidation at stresses considerably in excess of the existing in situ stresses in the ground (Nicholson and Jardine, 1981). The results obtained from this equipment have correlated accurately with results back-analysed from embankment behaviour (Fig. 12).

ACKNOWLEDGEMENTS

The road embankments described in this paper are part of Kent County Council road schemes and the authors are grateful to the County Surveyor, Mr. A.D.W. Smith, for his support in this work and for permission to publish this paper. Thanks are also due to the staff of Ove Arup and Partners and the KCC Highways Laboratory who jointly have carried out the work described in this paper.

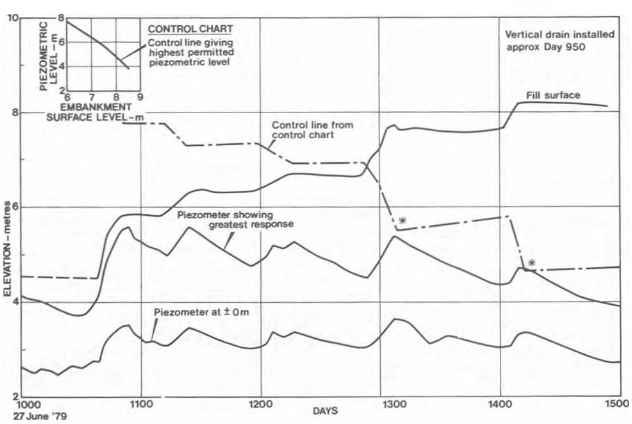

Fig. 15 Queenborough By-Pass. Progress of filling and control chart.

Table I
Predicted and achieved progress of filling.

SANDWICH					QUEENBOROUGH				
PREDICTED		ACHIEVED		Earliest Possible Day	PREDICTED		ACHIEVED		Earliest Possible Day
Depth of fill m	Day	Depth of fill m	Day		Depth of fill m	Day	Depth of fill m	Day	
3.8	Start	3.8	Start	Start	3.0	Start	3.0	Start	Start
4.3	1	4.3	12+	1	3.5	1	3.5	4+	1
5.1	25	5.5	84+	46	4.0	1	4.0	8+	1
5.9	75	6.2	160+	77	4.5	5	4.4	19+	1
6.7	135	6.9	206+	122	5.0	25	4.9	76+	1
7.5	210	7.5	367+	272	5.5	49	5.3	161+	64
8.3	290	8.2	418+	392	6.0	75	5.8	236+	176
9.1	360	9.1	487+	452	6.5	109	6.5	244*	236
		Complete			7.0	147	7.15	351*	351
					7.5	201	to be placed		
					8.0	300			

Notes
1. + These layers could have been placed earlier.
2. * These layers gave the maximum permitted response, see Fig. 15.

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