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PREDICTED AND OBSERVED PERFORMANCE
OF
MOTORWAY EMBANKMENTS
ON A
SOFT ALLUVIAL CLAY IN SOMERSET

A thesis submitted to the University of Surrey
for the degree of Master of Philosophy in the
Department of Civil Engineering

by

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M.I.Mun.E., M.I.H.E.

DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF SURREY

APRIL 1975
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The dimensions of the quantities in the following list are the ones generally used in the thesis and in practice. A number of the reports referred to were originally prepared in Imperial Units and these have been converted to SI units where possible.

Where Imperial Units have not been converted to SI Units the following conversions apply:

Length

1 ft = 0.3048m
1 in = 25.4mm

Mass

1 ton = 1.016 Mg
1 lb = 0.4536kg

Density

1 lb/ft³ = 0.157 kN/m³

Force

1 lb f = 4.448 N
1 kg f = 9.807 N

Pressure

1 lbf/ft² = 0.04788 kN/m²

Coefficient of Consolidation Cv

1 ft²/year = 0.0929 m²/year

A (m²) = Area of a mass of soil
A̅ = Pore pressure coefficient = Ud / Δ p
B = Pore pressure coefficient = Ua / p₃
b (m) = base width of embankment
c (kN/m²) = cohesion
c' (kN/m²) = cohesion intercept with respect to effective stress
Cc = compression index
Cc = weighted average compression index
Cv (m²/year) = coefficient of consolidation
D = depth factor
d (m) = depth of compressible material
\[ e_0 \quad = \quad \text{void ratio under effective overburden pressure } P_0 \]
\[ e_h \quad = \quad \text{horizontal strain} \]
\[ e_v \quad = \quad \text{vertical strain} \]
\[ F \quad = \quad \text{factor of safety} \]
\[ f_o \quad = \quad \text{correction factor} \]
\[ H \quad (\text{m}) \quad = \quad \text{depth of slope} \]
\[ H \quad (\text{m}) \quad = \quad \frac{1}{2} \text{ thickness of compressible layer} \]
\[ h \quad (\text{m}) \quad = \quad \text{height of embankment} \]
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\[ i \quad (\text{degrees}) \quad = \quad \text{inclination of Slope Indicator casing} \]
\[ k \quad = \quad \text{Constant relating to cohesion of soil (Bjerrum 1965)} \]
\[ k \quad = \quad \text{Slope Indicator constant} \]
\[ \Delta L \quad (\text{mm}) \quad = \quad \text{culvert joint extension} \]
\[ L \quad (\text{m}) \quad = \quad \text{culvert joint spacing} \]
\[ l \quad (\text{m}) \quad = \quad \text{average distance between Slope Indicator readings} \]
\[ M \quad (\text{mm}) \quad = \quad \text{total lateral movement of Slope Indicator tubing} \]
\[ m \quad = \quad \text{stability analysis coefficient} \]
\[ N \quad = \quad \text{Stability Number} \]
\[ N \quad = \quad \text{Number of readings per Slope Indicator installation} \]
\[ P_0'(\text{kn/m}^2) \quad = \quad \text{effective overburden pressure} \]
\[ P_{c}'(\text{kn/m}^2) \quad = \quad \text{effective preconsolidation pressure} \]
\[ \Delta p \quad (\text{kn/m}^2) \quad = \quad \text{applied pressure} \]
\[ P \quad (\text{kn/m}^2) \quad = \quad \text{Maximum unit load of embankment} \]
\[ qu \quad (\text{kn/m}^2) \quad = \quad \text{compressive strength of clay} \]
\[ R \quad (\text{m}) \quad = \quad \text{radius of failure circle} \]
\[ Re \quad = \quad \text{strain ratio} \]
\[ Su \quad (\text{kn/m}^2) \quad = \quad \text{undrained shear strength} \]
\[ s' \quad (\text{kn/m}^2) \quad = \quad \text{minimum shear strength} \]
\[ T_v \quad = \quad \text{Time factor} \]
\[ t \quad (\text{years}) \quad = \quad \text{time} \]
\[ u \quad (\text{kn/m}^2) \quad = \quad \text{pore water pressure} \]
\[ \% \quad = \quad \text{degree of consolidation} \]
\( \bar{U} \) \%(\) = Average degree of consolidation

\( U_s \) \%(\) = applied pore water pressure

\( \Delta \sigma_v' \) (kN/m\(^2\)) = vertical effective stress

\( x \) (m) = lever arm

\( z \) (m) = depth of compressible material

\( \alpha \) (degrees) = angle of base of slice

\( \gamma \) (kN/m\(^3\)) = bulk density

\( \gamma' \) (kN/m\(^3\)) = submerged density

\( \gamma_w \) (kN/m\(^3\)) = density of water

\( \Theta \) (degrees) = central angle

\( \Theta' \) (degrees) = angle of shearing resistance with respect to effective stress

\( \Theta_u \) (degrees) = angle of shearing resistance (undrained)

\( E_i \) (m) = initial settlement (Bjerrum 1972)

\( E_t \) (m) = total settlement (Bjerrum 1972)

\( \eta \) = efficiency factor (Bjerrum 1972)

\( \delta_c \) (m) = settlement at the end of primary consolidation

\( \delta_t \) (m) = total settlement ie primary and secondary

\( \delta \) (m) = settlement

\( \delta_M \) (mm) = change in lateral movement
ACKNOWLEDGEMENTS

The writer would like to express his gratitude to Dr N E Simons and Dr B K Menzies for the assistance and stimulation which they offered during this work, and to Mr J M McKenna for his general encouragement and help in assimilating the data obtained from the instrumentation.

The writer is also obliged to Mr P G Lyth, Director of the South Western Road Construction Unit for his permission to re-produce the results of this work and to Mr A S Turner, County Surveyor of Somerset and Chief Engineer of the Somerset Sub-Unit, South Western Road Construction Unit, for his general encouragement.

The writer would also like to thank amongst many others, Mr B A Heskins and Mr A R Croall, for their assistance in taking the many readings.

The writer would also like to acknowledge the patience and understanding shown by his wife and family during the course of the preparation of this work.
SUMMARY

For 35km, the Birmingham to Exeter Motorway, M5, crosses the soft alluvial clays of the Somerset levels. This area extends from 15 to 50km south of Bristol and comprises alluvial sediments to an average depth of 26m.

The sediments are so soft and unstable that the stability of the motorway embankments was low and large settlements were expected.

The embankments were surcharged and allowed to settle for one year in order to reduce the post construction settlement.

Instrumentation was installed at 24 locations along the motorway, to control the stability of the embankment during construction and to check the settlement and surcharge calculations. The stability
of the surcharged motorway embankments was checked by observation of ground movements and that of the side road embankments by ground movements and effective stress analyses, because the main embankment was much lower than the side road embankments.

The approach embankments to the side road overbridges, which reached a maximum height of 8.5m, were constructed over the higher section in pulverised fuel ash.

Even using this lightweight fill, construction on all side road embankments was halted prior to completion, and stage construction and berms were necessary to reach the designed height in several cases.

The observed performance of the motorway and side road embankments has been compared with the predicted performance, in order to assess the accuracy of the stability and the settlement and surcharge design calculations. The accuracy of these predictions has a considerable economic effect on the motorway expenditure in an area where the earthworks content of the work amounts to 30% of the total.

Methods of improving the foundation soils by sand drains, vibro-floatation or relief piles were not considered suitable for these ground conditions.
The design of the motorway was prepared by the Somerset Sub Unit of the South Western Road Construction Unit, for the Department of the Environment with Mr J M McKenna as Soils Consultant.
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CHAPTER I

INTRODUCTION

The M5 Motorway from Birmingham to Exeter forms part of the Government programme for a principal road network linking major development areas in Britain.

The route chosen for the motorway as shown in Figure 1 follows closely the existing A38 Leeds to Exeter, Trunk Road and the Great Western Railway line as designed by I K Brunel at the end of the 19th Century. The route provides a bypass for the towns of Highbridge, Bridgwater, Taunton and Wellington which are developing with the influx of light industry into the area. It also provides a link for the sea-side areas around Weston super Mare, which border the Bristol Channel.

The section of motorway from Clevedon, 10km south of Bristol to Huntworth 2km south of Bridgwater is built on the soft alluvial clay of the Somerset levels, which form a flat expanse of pasture land approximately \(5 \times 10^4\) hectares in area and is bounded by the Mendip Hills to the north and the Quantock Hills to the south.

For 35km the motorway crosses the levels where the depth of alluvium is typically about 26m. These sediments are so soft that the stability of all embankments was low and large settlements were expected.
PLAN OF THE MOTORWAY ACROSS THE SOMERSET LEVELS - FIG. 1.
The alluvial clay which is also evident on the Welsh side of the Bristol Channel, and is sometimes referred to as the Bristol Channel alluvium, has in the past presented many geotechnical problems relating to civil engineering works and these are referred to in more detail in Chapter 5.

On sections of the A38 regular maintenance is necessary to eliminate the effects of differential settlement and to maintain the smooth riding quality expected on a trunk road servicing the South West of England.

The Central Electricity Generating Board and the Somerset River Authority have both adopted piled construction for their pylons, river sluices and pumping stations, respectively. This minimises the affects of settlement and is regarded as an economical solution for small scale structures.

The alluvial clay is lightly overconsolidated as revealed from laboratory tests and may explain the absence of any settlement recorded on a 1.3m embankment constructed in 1966 for a new section of the A38 Trunk Road near Bridgwater. The degree of over consolidation approximates to the loading from an embankment 1.2m high built in 21.2kN/m³ material.

The problems facing the design engineers were: firstly, to design the motorway and side road embankments to ensure stable construction during the allowable period in the contract and,
secondly to ensure that the designed surcharge was sufficient to limit the post construction settlement on the motorway embankment to a design value of 150mm in the 50 years after the start of construction.

The rate at which the secondary consolidation occurs has an influence on the maintenance programme but it was envisaged that with flexible construction the life of the hot rolled asphalt wearing course would be 10 to 15 years which would mean three resurfacings before the end of the design period. This work would accommodate the differential settlement occurring along the motorway and particularly at the underbridge crossings which were piled.

Four trial embankments have been constructed on the alluvium at different times, one further up the Bristol Channel at Avonmouth in 1965 (Murray 1967 and 1971) and the other three on the Somerset alluvium in 1967, 1968 (Soil Mechanics Ltd 1968 and 1969). The trial embankment constructed at East Brent (1967) which is approximately 6.5km north of Highbridge, failed after reaching a height of 7.9m using fill of density 22.75kN/m³.

The maximum height of the approach embankment to the side road overbridges is about 8.5m and Pulverised Fuel Ash (PFA) from Aberthaw power station in South Wales was used to build the higher parts of these embankments. Even allowing for the use of PFA which is two thirds the density of common fill, the stability of every embankment had to be closely monitored during construction.
Twenty four monitoring locations were selected on the section of motorway between Edithmead and Huntworth, for the installation of instrumentation to record pore pressure, settlement, lateral movement and strain under the embankments. The instrumentation has been described in more detail elsewhere. (McKenna and Roy 1973), see Appendix VII.

The stratigraphy of the alluvium can change appreciably in a very short distance and it was unlikely that the piezometers installed would be positioned in the weakest ground and in addition they were placed under only one of the two approach embankments to the side road overbridges. It was therefore necessary to monitor ground movements using toe pegs and settlement gauges.

The recording and interpretation of the readings from the instrumentation for this section of motorway (referred to as Contract 12) was controlled from the motorway site office by a team of 8 engineers and technicians. The writer was in charge of the section and worked with the Soils Consultant Mr J M McKenna on the interpretative work and advised the Resident Engineer concerning the contractual implications of the findings.*

*The Contractor was restricted to placing 300mm of fill per day after reaching a height of 4m above original ground level. When the results of the instrumentation on an embankment indicated that the factory of safety had dropped to the design figure, he was issued with an order to cease filling. The filling operation was not restarted until the instrumentation indicated it was safe to proceed and then often at a reduced rate of construction. The specification required the Contractor to allow for these disruptions and for the variations in the rate of placing of fill material.
The section of motorway reported on here is that from Edithmead to Huntworth. The writer has examined the predicted and observed performance of the embankments on the soft alluvial clay and these are compared in order to assess the accuracy of the methods used in the calculation of the stability and settlements of the embankments.

Methods of improving the foundations for the embankments were considered, but rejected on the basis of the findings from the trial embankments at East Brent and Clevedon where vibrofloated stone piles and sand drains were tried without having any effect on the settlement performance of the foundations. A chapter dealing with methods of ground treatment is included and suggestions for further research studies have been made.

The design of the motorway was undertaken by the Somerset Sub-Unit of the South Western Road Construction Unit. The site investigation for the works was carried out by Soil Mechanics Ltd in conjunction with Mr J M McKenna, Soils Consultant who subsequently advised on the interpretation of the monitoring information during construction.

The pre-loading or surcharge design would appear to have been successful in restricting the post construction settlement in the first year of operation of the motorway to 25mm.
CHAPTER 2

SITE CONDITIONS

2.1 INTRODUCTION

The alluvial plain extends a considerable distance inland from the coast thereby rendering uneconomic any alternative route for the motorway skirting the fen area.

The chapter examines the topography, drainage and commercial interests along the selected route. The geological information from the published maps and memoirs is briefly outlined together with the geotechnical properties of the alluvial clay.

2.2 GENERAL TOPOLOGY

The Somerset levels is a flat expanse of pasture land with a surface elevation of 5.5 to 6.0m above ordnance datum (mean sea level at Newlyn, Cornwall). Until the turn of the century the majority of this area was covered by water at high tides. This inundation of water had been gradually reduced as a result of drainage works undertaken by the Churches of Glastonbury and Wells in medieval times and by subsequent attempts to reclaim land.

There is evidence to suggest that about the date 250 AD there was a rise in tidal level along the Somerset coast line which caused the flooding of large areas of land.
The reclamation and enclosure of the land was achieved by digging ditches which provided drainage for the fields as well as boundary markers and avoided the necessity for planting hedges.

This work established a vast network of ditches (or rhynes as they are known locally) which were later drained to three main waterways, the Rivers Brue and Huntspill and the Kings Sedgemoor Drain (Figure 1). These rivers are controlled by sluices and provide the necessary water storage for the summer months when the water is pumped into the rhyne systems from the rivers, to irrigate the land and provide drinking water for the cattle.

The history of the levels and the drainage works undertaken is described in more detail by Kelting (1968).

The alluvial sediments to the north of the Polden Hills overlie the Lower Lias bedrock and to the south they overlie the Keuper Marl as indicated in the sections shown in Figures 2 and 3. This is brought about by a northerly dip in the strata of about 4° which exposes the lower Triassic rocks as you move further south.

At the beginning of the twentieth century an exploratory borehole was put down at the base of the southern scarp of the Polden Hills to investigate the possibility of coal in the area. Instead at a depth of 180m, a 30m band of rock salt was encountered, which was subsequently worked for a number of years before being abandoned.
During the site investigation, borings were made to determine the effect of the salt workings on the overlying Marl and to see whether there had been any leaching of the rock salt in the area south of the Polden Hills. Here the salt band was believed to outcrop or be within a short distance of bedrock level, due to the 4° northerly dip on the strata.

The marl encountered was slightly brecciated with large quantities of selenite and calcite crystals but otherwise showed no signs of disturbance down to a depth of 46m which represented 28m penetration into bedrock.

2.3 GEOLOGY OF THE ALLUVIAL AREA OF SOMERSET

The alluvial flats which form the Somerset levels are the second largest area of fenland in England (see Figure 1).

According to the Institute of Geological Sciences, (Green and Welch 1965), the alluvial flats consist of Pleistocene to Recent estuarine alluvial sediments which were laid down in two major depositions. The first deposition from 6300 to 5000 BC was due to a relatively rapid rise in sea level and deposited initially sandy then grey estuarine clay with minor peat beds up to about ordnance datum (OD) level. There followed a long period of peat accumulation over a wide area which now forms the main peat bed referred to as the "OD" peat.
The second marine incursion at about AD250 deposited a blue-grey estuarine clay about 4.25m thick over the peat with a level surface at 5.5 - 6.0m above OD. This clay, the top surface of which is slightly dessicated, is sometimes referred to as the Romano clay.

The intermittent rise and fall in sea level gave rise to the thin peat beds in the lower grey clay as can be seen in Figure 4 and led to the deposition of fine sand or silt layers which form the lower laminated clays. The laminations are shown clearly in photographs A, B, C and D, of the air dried 54mm piston samples taken during the instrumentation sub-contract.

The southern part of the route of the motorway is divided by the prominent east-west scarp of the Polden Hills which presents a study of the Upper Triassic and Lower Jurassic successions from the Keuper Marl to the Lias clays and limestones. A typical section of the Lower Lias can be seen in Photograph E. This material formed the major proportion of the surcharge and embankment fill.

2.4 ENGINEERING PROPERTIES OF THE CLAY

Details of the soil survey investigations for the motorway are outlined in Appendix I, and a characteristic geotechnical profile is shown in Figures 5 and 6 for the sections north and south of the Polden Hills. The moisture contents of the clays vary between 25 to 85% with an average of 40-50% and from 110 to 360% for the peat.
### Ground level 4-43 above O.D.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.68</td>
<td>Firm brown and grey mottled CLAY with a little silt-becoming softer and more grey with depth.</td>
</tr>
<tr>
<td>3.28</td>
<td>Soft to firm light-grey CLAY with a little silt. With particles of organic matter, fine white rosettes and small root holes.</td>
</tr>
<tr>
<td>4.28</td>
<td>Compact black partially decomposed fibrous PEAT.</td>
</tr>
<tr>
<td>6.80</td>
<td>Soft light-grey CLAY with a little silt. Texture crumbly, fine white rosettes, remains of reed stems. Many small (5 mm dia) holes.</td>
</tr>
<tr>
<td>11.30</td>
<td>Soft medium-grey CLAY with some silt. Silty partings, approx. 1 mm thick in bands of 4 or 5&quot;, about 23-100 mm apart. A few peaty remains. A few vertical holes 1 mm dia.</td>
</tr>
<tr>
<td>16.10</td>
<td>Soft to firm dark-grey CLAY with occasional laminations of fine sand and partings of silt and fine sand laminaitions up to 3/8&quot;, mm thick spacing 12 mm to 150 mm. Partings, dimensions as above, vertical holes 1 mm dia.</td>
</tr>
<tr>
<td>17.08</td>
<td>Compact black-fibrous PEAT</td>
</tr>
<tr>
<td>24.10</td>
<td>Firm silty CLAY and silty fine sand in lenses.</td>
</tr>
<tr>
<td>29.90</td>
<td>Loose, to medium dense light grey silty fine SAND.</td>
</tr>
</tbody>
</table>

### TYPICAL BOREHOLE LOG - FIG. 4
PHOTOGRAPH A: Air dried 54mm diameter piston samples of the lower grey clay laid on polythene sheeting.

PHOTOGRAPH B: Air dried 54mm diameter piston samples of the lower grey clay laid on polythene sheeting.
PHOTOGRAPH C: Air dried 54mm diameter piston samples of the lower grey clay laid on polythene sheeting.

PHOTOGRAPH D: Air dried 54mm diameter piston samples of the lower grey clay laid on polythene sheeting.
PHOTOGRAPH E: Section of the Lower Lias clays and limestones

PHOTOGRAPH F: Hydraulic "push-in" piezometer tip (two tube)
<table>
<thead>
<tr>
<th>WATER CONTENT (%)</th>
<th>UNDRAINED SHEAR STRENGTH (kN/m²)</th>
<th>BULK DENSITY (kN/m³)</th>
<th>EFFECTIVE VERTICAL PRESSURE (kN/m²)</th>
<th>DESCRIPTION OF SOIL</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td>1</td>
<td>2</td>
<td>Fir. silty CLAY</td>
</tr>
<tr>
<td>1</td>
<td></td>
<td>2</td>
<td>3</td>
<td>Firm silty CLAY</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>3</td>
<td>4</td>
<td>Firm friable silty CLAY with traces of Peat</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>4</td>
<td>5</td>
<td>PEAT</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>5</td>
<td>6</td>
<td>Soft to firm friable silty CLAY with traces of black Peat</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>6</td>
<td>7</td>
<td>SAND</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>7</td>
<td>8</td>
<td>CLAY</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td>8</td>
<td>9</td>
<td>MARL</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td>9</td>
<td>10</td>
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<tr>
<td>15</td>
<td></td>
<td>16</td>
<td>17</td>
<td></td>
</tr>
</tbody>
</table>

Geotechnical profile of the alluvium — FIG. 6
south of the Golden Hills.
The clays are of intermediate to high plasticity as indicated in Figure 7 with plastic limits varying between 10 and 45% and liquid limits in the range of 30 to 60%. Peat samples gave values of plastic limit of between 100 and 300% and liquid limits of 130 to 340%. The limits for the alluvial clay generally plotted within ±6% plasticity index of the A-line on Casagrande's plasticity chart.

Values of preconsolidated pressure were determined from standard oedometer tests where the incremental loading was carried out with a load factor of 1.5 in some of the tests, the remainder being done at 2.0 (Casagrande, et al, 1965). The preconsolidation pressure was estimated using the Casagrande (1936) graphical method and the load-deformation (e-log p) curve was re-constructed to field conditions using the Schmertmann (1953) method.

*Dr A Casagrande proposed that on the rebound curve for an e-log p plot, a point was selected where the radius of curvature was a minimum and lines drawn (1) tangential to the curve at this point, (2) horizontal through the point and, (3) to bisect the angle formed by lines 1 and 2. Where the extended virgin compression line cuts the line formed in 3, represents approximately the preconsolidation load.

+Schmertmann proposed that the e-log p curve could be reconstructed by (1) plotting the point of existing effective stress and void ratio of the in-situ sample, (2) drawing a horizontal line, ie constant void ratio through this point, (3) estimate the preconsolidation load and draw a vertical line through the value.

His work also showed that straight line extensions of the initial virgin slopes of consolidation tests with varying degrees of disturbance intersect within a narrow void ratio and he suggested 42% of e as the best estimate for most clays.

Construct the geological re-bound curve parallel to the laboratory re-bound curve through point 1. Extend the initial laboratory virgin slope as a straight line until it intersects the 42% e line. Then estimate a recompression curve for the geological rebound through point 1 and line 3.
Plasticity Index

Liquid Limit

PLASTICITY CHART - FIG.7
The estimated preconsolidation pressures have been plotted against depth in Figure 8. The preconsolidation ratio is plotted in the lower half of this figure and from the results indicated by Bjerrum (1967) it has been concluded that the critical pressure increases with the effective overburden pressure at a ratio of 1.6 for the Somerset clays as with the Drammen clays, as shown by dotted line in Figure 8.

The clay generally has a water content at or just below the liquid limit for the higher liquid limit clays and generally above the liquid limit for the silty clays whose values range from 30 to 40.

Shear strength determinations indicated values of \( c' = 0 \) and \( \phi' = 32^\circ \) from consolidated undrained triaxial tests with pore pressure measurements (see Figure 10(a)). The values of the pore pressure parameter "A" (Skempton, 1954) recorded during the shearing of the specimens is shown in Figure 10(b).
Preconsolidation Pressures vs Depth - FIG. 8
TRIAXIAL TEST RESULTS

FIG. 10
3.1 INTRODUCTION

This chapter outlines some of the more important design features and constructional procedures for building embankments on soft alluvial clay. Restrictions were placed on the contractor in order to minimise disturbance of the dessicated clay crust which provided a protective cover for the underlying soft clays.

3.2 DESIGN FEATURES

The motorway embankment was designed to stand approximately 2m above existing ground level. The height chosen related to the following factors;

(i) At the three river crossings there was a minimum requirement for headroom under the under bridges*.

(ii) The area is still subjected to flooding in periods of high tide and heavy rain.

(iii) The motorway drainage had to discharge above ground water level.

*An underbridge referring to one carrying the principal road over an obstruction such as a road, river or railway.
McKenna (1973) indicated that the field coefficient of consolidation was of the order of 15m$^2$/yr, that is, not fast enough for most of the settlement to take place during the construction period, but fast enough for surcharge or preloading to be of benefit. The surcharge design involved constructing the 2m motorway embankment to an initial height of approximately 4m and allowing it to settle for one year in order to reduce the post-construction settlement* to 150mm in the 50 years after the start of construction.

The approach embankments to the side road crossings exceeded the calculated safe height of construction in heavy fill. The embankments were then designed to include lightweight fill (pulverised fuel ash)† over the higher sections and heavy fill elsewhere, as shown in Figure 11. The calculated safe height of an embankment made of heavy fill ($\gamma = 21.2$ kN/m$^3$) was 5.3m with 1 on 3 side slopes. The safe height of an embankment made of PFA ($\gamma = 14.14$ kN/m$^3$) fill was 7.9m with 1 on 2½ side slopes.

*Settlement of 150mm in 50 years is considered acceptable in terms of resurfacing the motorway. The maximum differential settlement being limited to 50mm allowed three resurfacings during this period, each of which approximates to the life of an asphalt wearing course.

†Pulverised fuel ash is a by-product of the coal fired conventional power stations. Its density ($\gamma$) is approximately two thirds that of heavy fill, namely 14.14kN/m$^3$).
It was recommended by J M McKenna (1970) that after placing 4m of fill the construction rate should not exceed 300mm of fill per day. This rate was chosen to allow time for the instruments to respond to the additional loading, and to allow time for the engineering site staff to record and analyse the results of the additional loading in terms of pore pressure, settlement and lateral movement. The 4m limitation on the rapid placing of fill represents 75% of the calculated safe height of construction and was selected to allow for the weaker pockets of clay.

The restriction referred to above did not however allow for the presence of the new drainage channels* (rhynes) adjacent to the motorway embankment. These rhynes, as shown in Figure 11, can vary in depth from 1-3m and are approximately 8m from the toe of the embankment. The net effect of the rhynes being to increase the embankment height by the depth of the channel, at the same time providing a small berm to the motorway embankment.

3.3 CONSTRUCTIONAL PROCEDURE

All trees and undergrowth were cut off at ground level and the roots treated with chemicals† in order to minimise disturbance to the ground.

*Drainage channels had to be provided to link the severed land drainage/irrigation ditches and also provide an outlet for the motorway drainage.

†All roots and stumps were sprayed with unformulated 245-T at a concentration of 1½ litres to 100 litres of diesel oil. Those stumps exceeding 300mm diameter and the stumps of species resistant to the above preparation were, in addition, painted with ammonium sulphamate at a concentration of 4kg to 10 litres of water.
The desiccated clay crust was left undisturbed in order to provide a degree of protection for the underlying soft clays. Accordingly, no surface soil was stripped from areas covered by embankment and no wheeled vehicle heavier than 1524 kg laden mass was allowed to run on the ground surface prior to placing 1m of fill. This prevented rutting and remoulding of the surface soil and a consequent drop in strength.

In order to allow two-way drainage for the alluvial clay, a 300mm thick drainage layer was placed over the area of embankments prior to placing fill. The drainage layer was designed to prevent material punching into the surface soil and consisted of a well graded non-cohesive material graded from 150mm to not more than 10% passing a 75 micron British Standard sieve size.

Tracked vehicles and wheeled vehicles less than 1524 kg laden mass were then used to construct embankments to 1m high, from which point there was no restriction on the plant to be used.

Some embankments were constructed using pulverised fuel ash (PFA) imported from Aberthaw Power Station, South Wales. On the motorway embankment it was economically preferable to use PFA to formation level, in order to reduce the embankment load and thereby reduce the amount of heavy fill surcharge required as shown in Figure 12. The motorway embankment north of the Polden Hills was constructed in this way, but to the south there was a surplus of the Lias clays and mudstones won from
the rock cutting through the hill and the embankment was constructed all in heavy fill.

The imported PFA was unloaded from railway wagons over a gantry on a specially constructed siding. After placing it was compacted by vibratory rollers. The Lias clays and mudstones were placed by dump trucks and spread and compacted by sheeps foot rollers.

The construction of embankments proceeded at a controlled rate of 300mm per day, after they had reached 4m high or at such a rate as directed by the Engineer. In the case of all the side road embankments construction was stopped before the total height of fill had been reached. A provisional quantity of surcharge was designed for these embankments, in order to minimise the post construction settlement, but in fact very little material was placed before construction was halted. It was possible during the one year settlement period to "top up" several embankments and a number of areas of the motorway embankment were treated, where the records indicated settlement in excess of the calculated value. These areas were identified early in the settlement period in order that any additional surcharge loading could be in place for an effective period of time.

Bridge construction, which involved piling most of the structures, was started at the end of the 12 months settlement period. This was specified in order to minimise the effect of
negative skin friction* on the driven piles and also to reduce the amount of lateral stress on the piles. The effect of these stresses on the pile designs is detailed in Appendix II. Bitumen coating was applied before driving to the section of the pile in the silty clays and the lower portion which penetrated the silty fine sands was left uncoated. This method was adopted following work done in Norway by Bjerrum, Johannessen (1965) and Bjerrum, Johannessen & Eide (1969) and is referred to in Appendix II.

In view of the amount of consolidation continuing after the year settlement period, it was decided not to use raking piles to carry the lateral forces on the structures but instead to strut the abutments by means of a rock fill layer at overbridges and by a concrete invert slab at the river crossings.

The Somerset Sub-Unit was responsible for the control of construction work, under the direction of the Resident Engineer Mr R Browning and subsequently Mr A J Billinghurst. The writer had a special responsibility to the Resident Engineer, during the monitoring of the construction of the embankments. On completion of this work he was responsible for the supervision of the piling sub-contract which involved both driven and bored cast in place piles for nine of the thirteen structures, on the lengths of motorway between Edithmead and Dunball.

*Negative skin friction is the downward force exerted on a pile due to the consolidation of adjacent soil.
CHAPTER 4

INSTRUMENTATION

4.1 INTRODUCTION

The object of the instrumentation was:

(i) to enable the actual vertical and horizontal movements of the foundations of the embankments to be measured in order to check the stability and the settlement and surcharge design calculations.

(ii) To measure the excess pore water pressure built up during construction and the subsequent dissipation during the settlement period. From these records the factor of safety of the embankments was calculated in terms of effective stress, and the settlement calculations were checked.

This chapter outlines the instruments used which are referred to in more detail by McKenna and Roy (1973), Appendix VII.

Selected piezometer groups, settlement gauges and slope indicators have been retained on site to provide a post construction record.

4.2 PIEZOMETERS

The low-air entry piezometer as shown in Photograph F was selected for use on the motorway, because of the range of
pore pressures expected and the response time involved. The standard piezometer tip was used, with modifications developed by Wilkes (1970)* which enabled the candle to be pushed into the ground in advance of boring and thereby avoid the disturbed zone of material.

Prior to the installation of any piezometers in a group a borehole was constructed and continuous 54mm piston samples were taken. These were extruded in the site laboratory and split and air dried for detailed examination and logging. The required positions of the piezometers in a group was determined, by reference to the stratigraphy revealed from this examination.

The borehole which had been used to obtain the piston samples was advanced far enough to penetrate the silts and sands underlying the clay, or to bedrock. Two way drainage had been assumed in the settlement calculations and in order to check that there was free drainage in the underlying sands a piezometer was installed in the borehole. The piezometer was placed in a sand pocket, formed in the borehole for a height of 1m and then backfilled with a 3:1 Bentonite/Cement grout, tremied into place. The design of the grout mix is discussed in Appendix III. The remaining piezometers in the group were then installed in 75mm

*Wilkes in conjunction with Soil Instruments Ltd developed a penetration head and reinforcing brass spindle for use with the standard filter candle.
diameter flight auger* boreholes and were pushed the last 300mm into undisturbed material, before being grouted up. This ensured that the clay structure around the piezometer tip had suffered the minimum of disturbance and enabled more accurate constant head permeability tests to be carried out. The permeability apparatus and an amended test procedure is outlined in Appendix IV.

The tests were conducted at three stages in construction in order to determine the variation in the coefficient of consolidation:

(i) prior to placing fill

(ii) after construction of the embankment

(iii) prior to removal of surcharge

The results from these tests indicate little change in the coefficient of permeability over this period, the value generally ranging between $10^{-7}$ to $10^{-9}$ m/sec.

The piezometers were installed in groups along the motorway as shown in Figure 13 and were connected to gauge houses, equipped

*Flight auger boring was used as it represented a 50% saving in cost over shell and auger boring.
TYPICAL LAYOUT OF INSTRUMENTATION ON THE MOTORWAY - FIG. 13
with de-airing apparatus. It was recognised soon after the initial de-airing and commissioning of the gauge houses, that the pressures applied in the de-airing process were wrong. The information supplied by the manufacturer assumed a gauge house at a lower elevation than the piezometer, e.g., a dam site. The calculation of the de-airing pressures was modified to avoid any change in pressure and disturbance to the soil around the tip. It was noticeable that the excess pore pressures fell sharply after de-airing by as much as a meter head of water.

The variability in the consolidation characteristics of apparently similar soils is illustrated in Figures 14 and 15 which show the performance of approach embankments to the north and south banks of the River Huntspill.

Performance records such as Figure 14 and 15 which are plots of excess pore-water pressure and settlement against the logarithm of time were prepared for all piezometer and settlement gauge installations and formed the basis of the assessment of stability, and settlement and surcharge performance. This will be discussed in more detail in later chapters.

4.3 ROD SETTLEMENT GAUGES

The gauges were installed at ground level at 100m intervals along the centre line of the motorway and 200m intervals at an off-set of 17.3m from the centre to provide two gauges at
EMBANKMENT PERFORMANCE RECORD NORTH BANK OF THE RIVER HUNTPHILL.

FIG. 14
EMBANKMENT PERFORMANCE RECORD SOUTH BANK OF THE RIVER HUNTSPILL.

Fig. 15
every 100m, as shown in Figure 13. The gauges were extended by the Contractor as the filling operations proceeded. Initially, protective walls were required to protect the gauges as shown in Photograph G but these were later dispensed with.

At the side road embankments 3 pairs of gauges were installed at 40m intervals over the high sections of the embankments.

### 4.4 PERFORATED SETTLEMENT GAUGES

Two perforated gauges were installed over each of the piezometer groups to allow the standing water level in the fill to be measured. A typical gauge is shown in Photograph H.

### 4.5 INDUCTIVE SETTLEMENT GAUGES

The Inductive or Electrical Vertical Settlement gauges as shown in Photograph J were installed in boreholes at four locations to measure the consolidation in the different strata. They were unfortunately prone to damage by the construction plant and did not yield any worthwhile information as a result.

Some results, however, from one of these instruments are shown in Figures 16 and 17 and indicate the increase in compressibility beneath the dessicated crust and the minimal change with depth.
PHOTOGRAPH G: Rod settlement gauge and protective barrier

PHOTOGRAPH H: Perforated settlement gauge
PHOTOGRAPH J: Inductive Settlement gauges

PHOTOGRAPH K: Horizontal settlement gauge
FIG. 17

INDUCTIVE SETTLEMENT GAUGE RESULTS - (contd)
4.6 HORIZONTAL SETTLEMENT GAUGES

These were installed under the embankments at 6 selected sites to measure the strain and settlement profile. The settlement gauge tubing and the recording instruments are shown in Photographs K and L.

The equipment was at this time still in the development stage and unfortunately no reliable readings of strain could be obtained. This information would have proved useful in checking the strain effects on the numerous culvert sections and a hot oil pipeline under the motorway embankments. The strain calculations as detailed in Appendix V are based on work by Cappleman (1967) and indicate an extension on each 1m precast culvert section of 95mm. At one stage during the construction, a collapse occurred at one end of a culvert causing the concrete headwall and one precast section to rotate and fall off.

Typical information obtained from these gauges is shown in Figures 18 and 19 and illustrate the settled profile of the embankment and the wide scatter of strain readings.

*Another use was found for the gauge when culvert profiles had to be checked. To level the soffit of the sections was difficult and involved a shortened pre-fabricated level staff and a torch, in order to obtain readings. The top of the culvert had been previously covered in the half width construction adopted. The gauge was set on a known datum level and the probe then placed on the soffit of each section and the settlement/level recorded.
PHOTOGRAPH L: Horizontal settlement gauge recording units

PHOTOGRAPH M: Slope Indicator Equipment (Wilson)
4.7 SLOPE INDICATORS

The Wilson Slope Indicator equipment as shown in Photograph M was used to monitor 20 installations essentially at bridge sites. The indicator tubing was installed in 150mm diameter boreholes constructed to give 1m penetration into bedrock. The holes were backfilled with a 3:1 Bentonite/Cement grout which was subsequently found to have a corrosive effect on the aluminium alloy tubing.

The locations for the tubing were selected to provide measurements of lateral movements at:

(i) a side road embankment near existing buildings,

(ii) a high approach embankment to the Huntworth Viaduct,

(iii) at all river crossings.

A tabulated computer printout was developed by B A Heskins to facilitate ease of conversion from electrical units to deflection readings in inches, this is detailed in Appendix VI.

The extent of the lateral movement was carefully monitored to

*On subsequent installations an epoxy resin coated tube was used to overcome this problem.

+B A Heskins was employed as assistant soils engineer on Contract 12 of the Motorway.
provide a basis for the design of the driven steel piles, as outlined in Appendix II, as well as providing a check on the stability of the embankments. This was achieved by examining the rate of maximum horizontal deflection against time and in relation to the embankment loading. An increase in the rate of deflection being anticipated prior to imminent failure.

Wilkes (1973) used the readings from inclinometer tubes as a means of control of the construction works on the Kings Lynn Southern Bypass. He proposes that by selecting a point at a level where maximum movement occurs and plotting the movement of this point \( \frac{\Delta s}{H} \) divided by the height of fill \( (H) \), against time \( (t) \), produces a linear plot for stable construction and where instability is imminent the rate of movement as expressed by \( \frac{\Delta s}{H} \) increases, as shown in Figure 20. The writer has attempted to interpret the results from the slope indicator readings at Withy Road embankment* by this method, see Figure 21. The increase in the rate of movement can be seen but it is regarded as unreliable as a means of control, as it does not relate the rate of deformation to the imminence of failure and it is important to relate accurately the mechanism of failure and the points of observation.

*This embankment reached a calculated factor of safety of 1.1, at the time filling was stopped, owing to the rapid increase in pore pressure.
HORIZONTAL DEFLECTION AT WITBY ROAD EMBANKMENT (after Wilkes 1972)

FIG. 21
FIG. 21(a)
4.8 LATERAL MOVEMENT PEGS

These were installed at 20m intervals 3m offset from the toes of embankments, where the height of fill exceeded 4m, as measured in relation to the adjacent drainage channels. This effectively included all embankment construction on the alluvial clays and represented an enormous amount of geodetic work. The pegs were referenced to survey base lines established 30m* from the toe of the embankment, to provide a reliable basis for the observations. The base line reference points were established in field boundaries to achieve some permanence. These were referred to at each set of readings by taping back 30m, to reference boards which were used as the theodolite site line, parallel to the base line. Peg offsets were related to this line.

*The failure of the East Brent trial embankment extended to 20m beyond the toe of the bank.
CHAPTER 5

EMBANKMENTS ON SOFT CLAY

5.1 INTRODUCTION

The stability of embankments constructed on the soft clay of the Bristol Channel alluvium, of which the Somerset Levels form a part, is considered in relation to the measurement of undrained shear strength.

Some of the factors affecting the measurement of undrained shear strength are considered together with the relationship between the mechanism of failure and the tests conducted to examine the critical shear strength of the soil along supposedly similar failure planes. This subject has been well documented in recent years by Parry (1971) and Bjerrum (1972) and the writer has attempted to relate their work to the Somerset alluvial clay.

5.2 LOCAL FAILURES ON BRISTOL CHANNEL ALLUVIUM

A number of local failures have been recorded of embankments constructed on the Bristol Channel alluvium and these tend to show an average value of undrained shear strength of approximately 21.6kN/m$^2$. 
5.2.1 Skempton and Golder (1948) reviewed two of these failures in their assessment of the accuracy of the \( \phi_u = 0 \) analysis. The first, a failure of a bauxite tip in Newport (Figure 22) occurred after 7.63m of material had been placed above ground level and caused the adjacent surface to heave about 0.916m. The minimum factor of safety found after analysing several slip circles using the \( \phi_u = 0 \) method of analysis, was 1.08. The calculated average shear strength of the clay is approximately 17.5kN/m\(^2\).

5.2.2 The second failure referred to occurred during the excavation of a river channel in 1940 near the village of Huntspill. The failure was initially referred to by the Building Research Station, who were asked to advise on the safe slope for excavation of the channel and also the safe height and distance from the top of the cut to deposit the spoil to form a flood bank. A section of the slip is shown in Figure 23 and an analysis of this section using the \( \phi_u = 0 \) method gave a factor of safety of 0.9. The calculated average shear strength was 16.3kN/m\(^2\).

Unconfined compression tests were conducted on samples from three test pits and two hand auger boreholes with varying results. The results generally ranged from
FAILURE OF A BAUXITE TIP AT NEWPORT (SKEMPTON AND GOLDER)

(Data from unpublished report of Soil Mechanics Ltd)
FAILURE OF A SECTION OF THE RIVER HUNTSPILL CUT

(Data from unpublished B.R.S Report)
11.03kN/m² to 16.55kN/m², but unfortunately, owing to the variation in the shear strength results and the uncertainty of the depth of the soft clay, no accurate analysis could be expected.

In the Newport and Huntspill analysis, the factor of safety has been calculated using the \( \phi_u = 0 \) method of analysis to an accuracy commensurate with the accuracy of measurement of the undrained shear strength. It should be mentioned however that this method of analysis relies on the clay being saturated and applies to the condition of no water content change with respect to the applied stresses at failure, a condition which would undoubtedly not apply to the more laminated silty clays of the Bristol Channel alluvium. Calculations by this method will not lead to a theoretically correct position of the shear surface and may over estimate the factor of safety by 20% to 30%.

As the basic assumptions imply the angle of shearing resistance must be \( = 0 \), but the true angle of internal friction will, in most cases, be appreciably greater than zero. The failure in the Eau Brink Cut also referred to by Skempton and Golder (1948) illustrates this point. Where the minimum factor of safety by the \( \phi_u = 0 \) analysis was found to be 1.02 but the critical circle using this analysis was appreciably
different from the actual slip surface and moreover the factor of safety along the actual surface was about 1.3.

5.2.3 In January 1968 a failure occurred in the trial embankment constructed at East Brent. The embankment had been constructed as the first stage in a programme of field trials for embankment construction on the Somerset alluvium. The intention was to construct the bank, as shown in Figure 24, to a height of 9.15m with a final crest width of 15.23m; this being the expected profile of the interchange embankment. The failure occurred after 7.92m of fill, of density 22.75 kN/m$^3$ had been placed and caused a 1.22m slump at the crest, the failure extending to 19.8m beyond the toe of the bank, as shown in Figure 25.

To prevent any further movement in the failed area and avoid additional damage to the instrumentation, a 15.22m berm 1.22m thick was placed along the toe of the bank.

Plastic movement in the clay foundation is thought to have been started as the shearing stresses increased with the bank height at 6.09m. An investigation of the slip by Soil Mechanics Ltd (1968) did not succeed in
TENSION CRACKS

DESIGN HEIGHT

BEFORE SLIP

AFTER SLIP

BERM PLACED AFTER SLIP

DISTURBED SAMPLES

CROSS-BEDDED STRATUM

PATHS

10

20

10

20

30

40

60

SCALE: 1 cm = 3 m
positively identifying the surface of failure, but there was sufficient evidence from boreholes and piston samples taken subsequent to the slip, to suppose that the failure surface extended below the peat bed, along one or two horizons of disturbed material.

Total stress analysis of the slip gave a calculated failure height of 6.09 and 7.92 for shear strengths of 21.6 and 27.8 kN/m². These represent the minimum average and average values estimated from the rather variable results of insitu vane tests conducted after the slip.

An effective stress analysis of the slip was not possible as it required a knowledge of the pore pressure distribution along the failure surface and as piezometers were only installed under the centre of the bank this knowledge was not available. Calculations were carried out with assumed pore pressure distributions and effective strength results from consolidated drained, undrained triaxial and shear box tests. These indicated values of $c' = 7.67\text{kN/m}^2$ and $\phi' = 28^\circ$ for the dessicated crust and $c' = 0$ and $\phi' = 28^\circ$ for the underlying soft clays.
The findings from the investigation indicated that there may have been a transfer of pore pressure along the peat bed from the centre of the bank without significant drainage. This potentially dangerous situation was investigated in the subsequent trials at Clevedon.

5.2.4 During the winter of 1968/69 two further trial banks, one of common fill (21.2kN/m³) and the other of pulverised fuel ash PFA (14.14kN/m³) were constructed for the South Western Road Construction Unit at Clevedon in North Somerset.

The embankments were positioned so that they would form part of the Clevedon interchange on the M5 Motorway. This fact subsequently limited the trials to a controlled construction experiment without risk of failure.

The banks were built on approximately 10m of alluvium as shown in Figure 26 and the intention was to construct both embankments continuously to a maximum height of 9.13m. The heavy fill embankment was built with 1 on 3 side slopes and had a 6.09 m wide berm on one side at a height of 4.57m. The lightweight fill embankment has 1 on 2½ side slopes. Both banks were designed to have a crest width of 15.22m at a height of 9.13m.
(a) HEAVY FILL EMBANKMENT

Scale: 1:500

Piezometer level at end of construction (day 130)

(b) LIGHT WEIGHT FILL EMBANKMENT

Piezometer level at end of construction (day 116)

305mm § Sand Drains at 1-03 m cc's

CLEVEDON TRIAL EMBANKMENTS (after Soil Mechanics Ltd 1969) FIG. 26
The construction of the two banks was carefully monitored and the results are set out in a report by Soil Mechanics Ltd (1969). Of the two banks, the one constructed in quarry waste, the heavy fill, showed signs of instability at a height of 6.84m, when construction was halted. After a shut down of 48 days, construction was resumed and an additional 1.3m of fill was placed before construction was again suspended and it was considered advisable to discontinue filling on this embankment.

Before construction of the pulverised fuel ash embankment started a row of sand drains were installed along one toe to examine the effectiveness of draining any pore pressures transferred along the peat layer, after East Brent (1968). The embanking operation then continued and reached the design height of 9.13m and filling was stopped.

Analyses of the embankments using an average vane strength result of 21.6 kN/m², gave calculated failure heights of 6.4m and 9.9m for the heavy and light fill banks respectively. The analyses were based on the assumptions that no drainage had occurred and there were no berms.
There was no evidence of the transfer of high pore pressures along the peat layer as suggested from the East Brent trial embankment.

5.2.5 There are a number of other failures of embankments and river cuts on the motorway which are not recorded at this time. It is however noted that at one site where a river channel cutting failed, the measured undrained shear strength was as low as 4.79 kN/m², thereby indicating the variability in the alluvial clays.

5.3 MEASUREMENT OF UNDRAINED SHEAR STRENGTH

In soft or sensitive clay soils it is difficult to obtain undisturbed samples for the determination of the undrained shear strength. The effects of disturbance caused by physical disruption of the sample and also by stress change have been investigated by a number of people and much of the work has been summarised by Davis and Poulos (1967). In clays of medium sensitivity such as the Somerset alluvium both sources of disturbance tend to reduce the shear strength, giving laboratory values below the field values.

In order to overcome the difficulty of obtaining undisturbed samples for the determination of the undrained shear strength, Cadling (1950) developed the vane test for in situ measurements
of the undrained shear strength. It was believed initially that the shear strength represented in these measurements was equal to the strength which could be mobilised in the clay when loaded, for instance, by an embankment, under undrained conditions.

As further studies became available (Golder and Palmer 1955; Casagrande 1960; Parry and McLeod 1967; Eide 1968; Ladd 1969) it was apparent that the measured value of vane strength was in general greater than the field strength and Casagrande attributed this difference to the effect of the rate of loading.

Bjerrum (1972) referred to 13 case histories of failures of embankments on soft clay, where the field shear strength could be reliably computed and where vane tests results were available. These are shown in Table I with the properties of the clays and the factors of safety computed from the results of vane tests.

From this study Bjerrum concluded that the discrepancy between the vane and the field shear strengths is larger, the more plastic the clay, and he related the factor of safety to plasticity as in fig. 27 to produce the correction factor (fig. 28) for the vane shear strengths. The average
<table>
<thead>
<tr>
<th>SITE</th>
<th>INDEX PROPERTIES %</th>
<th>FACTOR OF SAFETY</th>
<th>REMARKS</th>
<th>REFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scottsdale Embankment failure</td>
<td>140</td>
<td>150</td>
<td>42</td>
<td>108</td>
</tr>
<tr>
<td>Bangkok Test fill A</td>
<td>140</td>
<td>150</td>
<td>65</td>
<td>85</td>
</tr>
<tr>
<td>Bangkok Test fill B</td>
<td>140</td>
<td>150</td>
<td>65</td>
<td>85</td>
</tr>
<tr>
<td>Scrapsgate Embankment failure</td>
<td>70</td>
<td>112</td>
<td>30</td>
<td>82</td>
</tr>
<tr>
<td>Lancaster Test fill</td>
<td>120</td>
<td>120</td>
<td>48</td>
<td>72</td>
</tr>
<tr>
<td>Saint Andre de Cubzac Test fill</td>
<td>110</td>
<td>102</td>
<td>55</td>
<td>47</td>
</tr>
<tr>
<td>Matagami Test fill</td>
<td>90</td>
<td>85</td>
<td>38</td>
<td>47</td>
</tr>
<tr>
<td>Fornic Embankment failure</td>
<td>80</td>
<td>80</td>
<td>35</td>
<td>45</td>
</tr>
<tr>
<td>New Liskeard</td>
<td>53 - 47</td>
<td>60 - 55</td>
<td>24 - 27</td>
<td>36 - 28</td>
</tr>
<tr>
<td>King's Lynn Test fill</td>
<td>(70)</td>
<td>(60)</td>
<td>(25)</td>
<td>(35)</td>
</tr>
<tr>
<td>Palavas Embankment failure</td>
<td>64</td>
<td>64</td>
<td>32</td>
<td>32</td>
</tr>
<tr>
<td>Narbome Test fill</td>
<td>34</td>
<td>37</td>
<td>21</td>
<td>16</td>
</tr>
<tr>
<td>Portsmouth NH Test fill</td>
<td>50</td>
<td>38</td>
<td>22</td>
<td>16</td>
</tr>
<tr>
<td>Fair Haven Embankment failure</td>
<td>42</td>
<td>37</td>
<td>21</td>
<td>16</td>
</tr>
</tbody>
</table>

**FAILURES OF EMBANKMENTS ON SOFT CLAY**

- Crack through fill material
- Full shear strength mobilized in fill material
FIG. 27 Theoretical factor of safety at failure of embankments on soft clay plotted against plasticity index of foundation clay.

\[(g_u)_{\text{field}} = (g_u)_{\text{vane}} \cdot \mu\]

FIG. 28 Principle of analysis of the stability of embankments and footings on soft clay based on corrected vane shear strength.

[After Bjerrum 1972]
plasticity of the Somerset clays indicates a correction factor of 0.9 which accounts in part for the inconsistencies in the vane and unconfined compression strength results.

The value of undrained shear strength to be used in the calculation of stability of the embankments on the motorway was a matter of judgement as the unconfined compression test results from 54mm diameter samples were significantly lower than the Geonor Vane test results, the average minimum values being 16.8 and 31.2 kN/m² respectively, see fig. 29.

A study of the values of undrained shear strength produced by the vane and unconfined compression tests, from a number of case records (Golder and Spence 1958; Brown and Paterson 1964; Eide 1968; Ladd, Aldrich and Johnson 1969; Parry 1971) show strength ratios from 1.5 to 2.0, where the strength ratio is expressed as the vane shear strength divided by the unconfined compression shear strength. The writer has plotted values of strength ratio against the plasticity index, in fig 30, for the borehole on the North bank of the River Huntspill and there is some evidence to suggest that the ratio increases with the plasticity of the clay.

Some of the factors which are known to influence significantly the measurement of the undrained shear strength of clays have
UNDRAINED SHEAR STRENGTH $\text{kN/m}^2$

Unconfined Compression Tests

Vane Tests.
been summarised by Parry (1971) and Bjerrum (1972) and include:-

(i) Sample disturbance  
(ii) Influence of strain rate  
(iii) Anisotropy  
(iv) Structural features  
(v) Stress redistribution in embankment  
(vi) Progressive failure

5.3.1 SAMPLE DISTURBANCE

It is impossible to obtain a completely undisturbed sample since the act of sampling must disturb the soil to some extent.

The principal causes of disturbance are:-

(i) the boring process  
(ii) driving the sampling tool  
(iii) withdrawing the sample tool and  
(iv) the stress relief on the soil.

The action of the shell or clay cutter in constructing the borehole causes considerable disturbance, which can extend as far as four times the diameter of the borehole, below the bottom of the borehole. This effectively rules out the use of open drive sampling* in soft/firm clays, where the level of

*Open drive sampling referring to the use of an open ended sampling tube usually 100mm diameter, driven into the clay soil and the base of the sample is sheared by rotating the sampler prior to extraction.
disturbance becomes unacceptable and usually results in the loss of the sample.

In order to minimise disturbance, 54mm and 100mm diameter piston samples were taken in the alluvial clay. The piston sampler, as developed in Norway, has been described in more detail in Appendix I.

After taking the piston samples, it is important to ensure that the samples are sealed and are transported in a vertical position corresponding to the orientation of the sample prior to extraction. During the soil survey it was considered that one of the reasons for the low unconfined compressive strengths, compared with the insitu vane results, was due to disturbance of the sample during transit from the boring rig to the site laboratory. Two further boreholes were constructed adjacent to a previous boring and continuous samples were taken, extruded and tested on the spot, in the unconfined compression apparatus. The additional tests gave very similar results to those obtained in the original borehole and it was concluded that no significant disturbance had occurred during transit of the sample. With all the precautions taken to ensure the minimum of disturbance, it was still noticeable in the laminated clays that after having been split and laid out to air dry some samples showed signs of disturbance on the outer 10mm. This
took the form of a slight curvature in the silt laminations across the disturbed zone.

Brown and Paterson (1964) found that samples trimmed down from 50mm Shelley and 75mm piston samples gave higher values of undrained shear strength, than the 38mm diameter specimens jacked from the larger samples. This would serve to indicate the degree of disturbance which may be caused by poor sampling techniques.

5.3.2 **INFLUENCE OF STRAIN RATE**

This aspect of soil behaviour has been recognised (Casagrande 1960; Eide 1968; Bjerrum 1972), as an important factor to take into account when using the results of insitu vane tests or unconfined compression and undrained triaxial tests.

Casagrande (1960) observed that in unconfined compression tests of five minutes duration a sharp peak occurred in the plot of shear stress against shear strain. By comparison samples tested at constant water content in long duration triaxial tests gave strength values up to 30% lower than the unconfined tests and with much flatter stress/strain curves. The peak values in the unconfined tests occurred at 1% to 2% strains while in the long duration tests, peak values were reached at about 6% strains and with plastic deformation at practically constant resistance for much higher strains.
Bjerrum, Simons and Torbaå (1958) concluded that the effect of the strain rate on the undrained shear strength is due to an increase in pore water pressure as the time to failure increases.

It is evident from Bjerrum (1972) that the rate factor increases with the plasticity of the clay, as does the cohesive component of the shear strength. Therefore, a vane test in which the failure time is of the order of 10 minutes, cannot be reliably used to assess the short term stability of cuttings or embankments where the shear stresses leading to failure may be gradually applied over a period of many weeks.

The effect of time on the shear strength of soils is not fully understood and may be due to structural features in the soil or pore pressure behaviour. In order to overcome these problems it would appear necessary to conduct the unconfined and undrained triaxial tests at a rate to failure of at least 20 minutes, after Parry (1971). This rate should also apply to field vane tests and would mean rates of strain of the order of 0.01°/sec as compared with 0.1°/sec as BS 1377.

5.3.3 ANISOTROPY

There are essentially two types of anisotropy which may affect the undrained strength of a clay.
(a) Structural anisotropy
(b) Stress anisotropy

Structural anisotropy is the tendency for clay particles to align at right angles to the direction of the major principal stress during deposition and subsequent consolidation. This preferred orientation of the clay particles may present a weakness not exhibited in triaxial tests on vertical specimens but it should be apparent in shear box tests.

The type of laboratory test has to be carefully related to the failure condition being examined in the field in order to achieve a degree of similitude. When studying embankment stability Bjerrum (1972) considered the following types of laboratory tests most relevant to the condition being examined, namely, a compression test under the embankment itself, a direct simple shear test under the toe, and an extension test outside the toe as shown in fig. 45.

Bjerrum also compared the results of these tests, divided by the effective overburden pressure, with in situ vane tests and it is of interest to note that the vane results corrected to the same rate of strain as the triaxial tests are roughly equal to the average of the three types of laboratory test and this may explain why it is possible to apply vane tests to compute the stability of embankments. Bjerrum concluded
that the effect of structural anisotropy is greatest in the lean clays and reduces with increasing plasticity of the clay.

Attempts have been made to examine the effects of structural anisotropy and particle orientation on undrained shear strength by anisotropically consolidating a block of clay. Samples are then taken from the clay at different angles to the major principal stress during consolidation. Results of this work and others is summarised by Parry (1971) and generally indicates conflicting results. The expectation being that samples taken at 45° to the major principal stress should give the lowest strengths, but the results show a considerable scatter and "it is difficult to reach any definite conclusion other than that the clay is behaving predominantly in an isotropic manner."

Aas (1967) found from a study of the marine clays around the Oslo Fjord, using varies of different dimensions, that the ratio of the undrained shear strength acting along horizontal and vertical failure surfaces varied from 1 where the clay was slightly overconsolidated, to between 1.5 and 2 where the clay was almost normally consolidated.

At other sites around the Fjord in normally consolidated clays the shear strength along a 45° inclined failure surface,
measured with a special vane, was found to lie between the horizontal and vertical shear strengths.

Bjerrum (1972) concluded that the variation of strength with the direction of shear is not a significant factor in the measurement of undrained shear strength for clays of medium to high plasticity.

It would appear reasonable to assume that for a plastic clay, which tends to be less anisotropic, that the vane shear strength represents the average along a full semi-circular slip surface and that anisotropy is a factor of secondary importance in explaining the difference between the vane and field strengths.

A limited number of vane tests were conducted by the writer, using the Farnell vane test equipment* and 100 x 50mm and 50 x 100mm vanes, to study the anisotropy of the shear strength properties of the Somerset clay. The few results obtained were inconclusive in trying to compare the variation in the horizontal and vertical shear strengths. They did however indicate that a greater degree of rotation was

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*This piece of equipment is usually used in the bottom of a borehole and is manufactured by Leonard Farnell Ltd. The rod attached to the vane has a 70° free turn before engaging the vane, in order to enable the rod friction to be measured. The torque is applied at ground level by a Tripod mounted torque head.
necessary with the 50 x 100mm vane to reach peak shear strength, which also occurred after a greater interval of time. This effect was noted by Aas (1965) in his work on the marine clays of the Oslo Fjord.

The anisotropy of the initial state of stress in the ground as exhibited by Ko, the coefficient of earth pressure at rest, may vary from 0.5 for a soft normally consolidated clay to 2.0 for a stiff, overconsolidated clay.

5.3.4 STRUCTURAL FEATURES

Structural features such as fissures, joints, and faults are not usually observed in soft clay deposits except in the surface crust. In a number of boreholes the soft clay exhibited brittle fracture and was crumbly, with occasional fissuring, see photographs A and B.

Eide (1968) attributed the effects of fissuring in the Bangkok clay, on the overestimate of the measured undrained shear strength, as compared with the shear stresses producing failure beneath the embankments. The analyses of failed sections of embankment revealed F.O.S. of 1.5 to 2.0 using values of undrained shear strength determined by vane testing.

High shear strengths were recorded in the surface crust of
the Somerset alluvium, where dessication has occurred, and is shown in fig 29.

5.3.5 STRESS REDISTRIBUTION IN EMBANKMENT

Parry (1971) has discussed the formation of high lateral stresses in an embankment as a result of settlement of the embankment. Parry found that the stress redistribution which occurs within the embankment with increasing settlement, gives rise to a reduction in the vertical pressure below the centre of the embankment and causes the point of maximum vertical pressure to move to about mid-way between the embankment centre line and the toe.

The high lateral stresses induced by stress redistribution are not sustained if tension cracks develop or if lateral movements occur. The high lateral stresses are therefore only likely to occur during the early movements of a slip and may be responsible for the initial movements.

There is evidence from one embankment on the Somerset clay that a tension crack developed prior to the rapid build up of pore pressures under the embankment. The tension crack was exposed to a depth of 3.5m and indicated a gradual widening with depth. It is considered that tension cracks developed in most of the embankments, but they were not apparent due to the nature of the surcharge material,
namely Lias clay fill. Where the tension crack was evident the filling material was P.F.A. overlain by sand.

The settlement and strain profiles as shown in figs 18 and 19 indicate the amount of deformation occurring under the higher side road embankments, which it is considered causes the development of the tension cracks.

5.3.6 PROGRESSIVE FAILURE

Progressive failure is at present one of the relatively unknown factors which may influence the comparisons between vane and field undrained shear strengths.

Progressive failure is most likely to occur with highly sensitive clays (ie quick clays) or in soils which exhibit brittle behaviour, that is in soils where there is a sharp drop in strength once the maximum value has been reached. Casagrande (1960) observed that the sharp drop in strength recorded from the rapid unconfined tests was more an indication of the incorrect rate of testing, than of progressive failure.
CHAPTER 6

STABILITY ANALYSES

6.1 INTRODUCTION

In traditional embankment stability analyses it is assumed that the soil strength is reduced by a factor of safety until limiting equilibrium occurs. The embankment fails according to some plastic mechanism, commonly taken to be rotation on a circular arc.

Embankments built on the Somerset clay were designed so that there would be no risk of a shear failure in the clay causing a disastrous collapse. The value of the safety factor was assumed to be a minimum at the end of construction before any appreciable consolidation of the clay had taken place. The shear strength used in the initial stability analyses was accordingly the undrained shear strength of the clay.

This chapter compares the total stress and effective stress analyses used in the calculation of the factors of safety of the embankments and indicates some of the shortcomings in the methods employed.

During the construction of each embankment, the factor of safety was calculated in terms of effective stress from the results of a general stability analysis carried out by
J M McKenna using the Little and Price (1958) computer programme. The results of the general analysis were used to prepare stability design charts as shown in Fig 33, which enabled a rapid interpretation of the magnitudes of settlement and pore pressure in terms of stability. The derivation and use of these charts is given in more detail in this Chapter.

6.2 TOTAL STRESS ANALYSES

The short term end of construction stability problem can in principle be analysed by the total stress ($\sigma_u = 0$) analysis using the undrained shear strength of the soil. The reliability of this method of analysis in calculating the factor of safety depends almost entirely on the accuracy with which the undrained shear strength of the clay can be measured.

It can be seen from Chapter 5 that there are many factors which can influence the field behaviour of soft clays under loading. The most significant of these being anisotropy, testing rate and progressive failure. The effect of these factors being to reduce the value of the measured shear strength, whereas the effects of sample disturbance and in situ testing, tend to raise the measured value.

The amount by which each factor may lead to an overestimate of the factor of safety based on a $\sigma_u = 0$ slip circle analysis has been suggested by Parry (1971) and from these it can be seen that calculated factors of safety as high as 1.5 or 2
might be obtained where, in fact, failure was imminent.

Skempton and Golder (1948) and Bishop and Bjerrum (1960) published case studies of the end of construction or \( \Phi_u = 0 \) analyses of failures of footings and fills on saturated clay foundations. In tables II and III the calculated factors of safety obtained by total stress analyses are shown and indicate a narrow range of 0.9 to 1.15. It was concluded at that time that no particular problem arose concerning the prediction of the stability of embankments and footings on clay.

Recent work summarised by Parry (1971) and Bjerrum (1972) indicate that it is necessary to consider carefully the influence of factors affecting the measurement of the undrained shear strength.

On the Somerset clay the undrained shear strength was measured from 54mm piston samples tested in the unconfined compression tester. In situ Vane tests were also taken at 0.76m intervals using the Geonor vane tester.

The results obtained showed a significant disparity, with unconfined shear strengths generally 50% less than the in situ strengths indicated by the vane. It is recognised that the mode of failure being modelled by the two tests is completely
<table>
<thead>
<tr>
<th>Location</th>
<th>Approximate Average Properties of Clay</th>
<th>Factor of Safety</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Foundation Failures</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Model Footings</td>
<td>74</td>
<td>270 lb/ft²</td>
<td>1.00 Laboratory tests.</td>
</tr>
<tr>
<td>Kippen</td>
<td>70</td>
<td>350</td>
<td>0.95 Adhesion on sides of footing neglected.</td>
</tr>
<tr>
<td>Newport</td>
<td>60</td>
<td>375</td>
<td>1.08 Deep slip surface.</td>
</tr>
<tr>
<td>Gosport</td>
<td>80</td>
<td>250</td>
<td>0.93 Strength too low owing to slight sample disturbance.</td>
</tr>
<tr>
<td><strong>Critical height test</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>80</td>
<td>280</td>
<td>1.05 slip plane at 51°, not 45°.</td>
</tr>
<tr>
<td><strong>Failures of Clay Slopes</strong></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Chingford</td>
<td>145</td>
<td>300</td>
<td>1.05 Analysis based on observed slip surface.</td>
</tr>
<tr>
<td>Eau Brink Cut</td>
<td>45 &amp; 100</td>
<td>470</td>
<td>1.02 $\phi = 0$ slip surface different from observed.</td>
</tr>
<tr>
<td>Brightlingsea</td>
<td>90</td>
<td>240</td>
<td>0.99 Average shear strength not known with accuracy.</td>
</tr>
<tr>
<td>Waltham Abbey</td>
<td>80</td>
<td>650</td>
<td>1.06 Partly clay fill, but with very small air voids.</td>
</tr>
<tr>
<td>Huntspail River</td>
<td>75</td>
<td>310</td>
<td>0.9 Only approximate analysis possible; not a critical check.</td>
</tr>
<tr>
<td>Belfast</td>
<td>67</td>
<td>300</td>
<td>1.15</td>
</tr>
<tr>
<td>Greenock</td>
<td>40</td>
<td>260</td>
<td>0.95</td>
</tr>
</tbody>
</table>

**Practical Examples of the $\phi_a = 0$ Analysis of Stability of Clays**

(After Skempton and Goldsb 1948)
### TABLE III

1. Footings, loading tests

<table>
<thead>
<tr>
<th>Locality</th>
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2. Fillings

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**END OF CONSTRUCTION FAILURES OF FOOTINGS AND FILLS ON A SATURATED CLAY FOUNDATION (AFTER BISHOP AND BJERRUM 1960)**
different. In the first case the specimen is subject to a
vertical compressive force, which may relate to the field
condition over the section of the failure circle which is
nearly vertical. In the second case a prescribed cylinder
of soil is rotated until failure occurs. This mode of
failure is not found in any slip circle, but from experience
it has been found to represent the average undrained shear
strength around a failure circle.

The average minimum values of undrained shear strength for
the unconfined and vane tests were 16.8 and 31.2 kN/m$^2$. The
calculated average shear strength from embankment failures on
the Bristol Channel Alluvium was 21.6 kN/m$^2$ and this figure
was adopted in the analysis of the embankments on the
motorway.

In the total stress analysis the undrained shear strength $S_u$
is assumed to be constant around the slip circle. It is
further assumed that $S_u$ is reduced by the same amount all the
way around the slip circle to $S_u/F$ such that the failure
mechanism is brought into limiting equilibrium, as shown in
Fig 31.

Moments about the centre of rotation to give limiting
equilibrium are:
Disturbing Moment = Restoring Moment

\[ \text{ie } A \cdot X \cdot y = \left( \frac{\text{Su} \cdot R \cdot \Theta}{F} \right) R \]

\[ \therefore \frac{\text{Su}}{F \cdot x} = \frac{A \cdot x}{R \cdot \Theta} \]

where \( A = \) area of the disturbed mass (m²)

\( R = \) radius of the failure circle (m)

\( x = \) distance of centroid of the mass from the centre of the failure circle (m)

\( \Theta = \) angle described between the top and toe of slope (°)

\( \gamma = \) bulk density of soil (kN/m³)

\( \text{Su} = \) undrained shear strength (kN/m²)

\( F = \) factor of safety

The analysis has been further simplified by Taylor (1948) who produced a useful chart giving stability numbers for various values of depth factor and angle of slope. The geometrical factor \( \frac{A}{R \cdot \Theta} \) depends exclusively on \( H, D \) and \( i \),

where \( H = \) depth of slope

\( D = \) depth factor

\( i = \) angle of slope

and the "Stability Number" \( N = \frac{\text{Su}}{F \cdot \gamma \cdot H} = f(iD) \)

This form of analytical solution cannot be successfully applied to the embankments on the Somerset clay as it presupposes that,
the soil is perfectly homogenous below the toe of the slope and that the clay soil rests on a stiffer stratum which is not penetrated by the surface of sliding.

The major side road embankments have been analysed using total stresses in the soft foundation clay and an effective stress approach for the embankment fill, see Appendix VIII. The porewater pressure in the fill was assumed to be zero, based on the information available from the Perforated Settlement gauges.*

Particular attention has been paid by Parry (1971) and Bjerrum (1972) to the problem of how much the shear strength of the fill contributes to the stability of the embankments. Where incipient failure was preceded by the formation of a crack in the fill, as on the Westonzoyland Road embankment (see Fig 1), then the strength of the fill should be ignored.

The results of the total stress analyses, shown in Table IV, have been prepared, considering the strength of the fill and also assuming a tension crack to have developed prior to incipient failure. It is evident that in the case of the

*The Perforated Settlement gauge is detailed in Appendix VII. The purpose of the gauge was to measure the water level in the fill and with the exception of two gauges the water level did not rise above 0.6m from the bottom of the gauge. Thus indicating that the 0.3m drainage layer placed under the embankment had been effective in providing a drainage path.
<table>
<thead>
<tr>
<th>Site</th>
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<th>Factor of Safety</th>
<th>General Analyses</th>
<th>Remarks</th>
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</table>

*( ) Bjerrum (1972) corrected vane strength results.

**COMPARISON OF RESULTS FROM STABILITY ANALYSES**

**NC** No allowance for a tension crack.

**TC** Tension crack allowed throughout the depth of the embankment.
Westonzoyland Road embankment where a tension crack was apparent and yet the embankment remained stable, the analysis underestimates stability by at least 7%.

The total stress analyses shown in Table IV indicate a degree of disparity with the effective stress analyses which appears to be contrary to the trends suggested by Parry (1971). With the exception of Newbridge Lane, the total stress analyses underestimate stability according to the effective stress results, by an average 12%. Parry (1971) indicates that the effective stress analyses achieve a reduction in the factor of safety and are more reliable than the $\phi_u = 0$ analyses providing realistic values of pore pressure can be obtained.

This disparity may arise because of the margin of error in the selection of the average shear strength data. If the conversion factor ($f^\alpha$), as suggested by Bjerrum (1972), is applied to the minimum average strength indicated by the vane, this produces a value of undrained shear strength of 26.5kN/m$^2$. The factors of safety obtained using this value of the undrained shear strength are shown in brackets in Table IV and with the exception of Newbridge Lane and Withy Road produce a better correlation with the effective stress analysis results. The average vane strength in the Withy Road area is lower than the average for that section of motorway and may account for the poor correlation.
The Newbridge Lane analysis is at variance with these results and a possible explanation may lie in the fact that this is the only side road embankment running parallel to the motorway, prior to crossing.

The construction of all the embankments on the motorway was achieved without a disastrous collapse and would therefore indicate factors of safety in excess of unity. The total stress analyses of the embankments, using the adjusted vane shear strengths, indicate that in all conditions except that of the New Road embankment, where the strength of the fill is neglected, the embankments are stable.

The Westonzoyland Road embankment was constructed in PFA and sand until the filling was stopped and a crack was observed in the fill.* The other embankments had one or more layers of heavy fill which obscured any cracks in the embankments.

Embankment construction on all side roads was stopped at the onset of incipient failure as indicated by a rapid build up of pore water pressure and settlement in relation to the applied load, and as shown in Fig 32(a) and (b). The general stability charts, referred to later in this Chapter, were then used to calculate the factor of safety of the embankments.

*The crack referred to was excavated to 3.5 metres and indicated an increase in width from 7mm to 60mm with increase in depth. The narrowest point occurred 0.9m below the top of fill.
FIG 32a - Settlement vs Applied Load

FIG 32b - Pore Water Pressure vs Applied Load
which are shown in Table IV.

A number of embankments had additional fill placed in order to reach the design heights. This operation was permitted when the instrumentation indicated it was safe to proceed.

In the case of the Westonzoyland Road embankment the pore water pressures were dissipating very slowly and it was evident that, within the Contract period, it would not be possible to place the additional fill required to achieve the design height. This necessitated a redesign of the motorway profile through this section and by raising the profile and providing wide berms adjacent to the side road embankments construction was completed.

The calculations for a total stress analysis of one of the side road embankments is set out in Appendix VIII (A).

6.3 EFFECTIVE STRESS ANALYSES

Effective stress calculations using the Bishop Simplified Analysis have been carried out, with measured porewater pressures. The potential failure surfaces investigated have passed through the zones of maximum pore water pressure, and are all assumed to be circular arcs.

*A berm refers to an additional thickness of fill placed adjacent to an embankment to provide loading at the toe of the embankment and thereby improve stability.*
The results of these analyses are listed in Table IV and a typical calculation of a side road embankment is detailed in Appendix VIII (B).

In analysing the stability of embankments on the Somerset clays, the resisting force provided by the shear strength of the embankment fill has been ignored. This is justified where vertical longitudinal cracks form in the central part of the embankment, such that a slip surface could follow them without meeting any resistance. There are cases recorded where longitudinal cracks have developed after the construction of the embankment and during the subsequent period of consolidation, causing failure in the embankment (Bjerrum 1972). The vertical settlement of an embankment induces straining in extension under the centre of the loaded area as shown in Figs 18 and 19. Where the fill is slightly cohesive, this may result in a crack developing at the base of the embankment and extending over the full height of the fill, as observed in Westonzoyland Road embankment.

Four of the effective stress analyses shown in Table IV gave factors of safety greater than those given by the total stress analyses. Parry (1971) refers to two case records (Marsland, 1957 and Parry, McLeod, 1967) where the effective stress analyses based on measured or reliably computed pore pressures gave reduced factors of safety compared with the total stress.
analyses and were regarded as more realistic.

One of the most significant factors affecting the measurement of the undrained shear strength, namely the rate of strain, has a much smaller effect on the effective shear strength parameters measured in the drained triaxial compression test than on the undrained shear strength results from the quick undrained, unconfined compression or vane tests. The strain rate effect is much less, due to the very slow rate of testing required to establish the $c'$ and $\phi'$ parameters in the drained test. The peaking observed by Casagrande (1960) in the shear stress against shear strain plots for unconfined compression tests of five minutes duration, should not occur in the drained test, which may last three days.

The two factors which have a marked influence on the accuracy of the effective stress analysis are the assessment of the cohesion intercept $c'$ and the pore pressures in the soft clay. The Somerset clay is lightly overconsolidated and from the results of the consolidated undrained triaxial compression tests with pore pressure measurements, shown in Fig 10(a) the $c'$ value for the soft clay has been assumed to equal zero. In the dessicated crust a value of $c' = 7.66$ kN/m$^2$ has been used.

The effective stress analysis detailed in Appendix VIII(b) use measured values of pore pressure from the centre line, hard shoulder and toe piezometer profiles, shown in Fig 11. The
manual analyses are based on a simplified section of the embankment and consider a limited number of soil properties throughout the foundation strata. This is done in order to restrict the analyses to workable proportions and yet retain the degree of accuracy required of such an analysis.

In order to verify results obtained by the manual analyses a number of embankments were analysed using the Geocomp Slip Circle/1 computer programme, outlined in Appendix VIII. The programme enables the user to input a detailed section of the embankment, with measured pore pressures expressed in terms of pore pressure ratio (ru) or excess pressures.

The original computer runs experienced difficulties in finding an acceptable system of co-ordinating the section of the embankment. The programme was initially written for the analysis of cut slopes.

The results of the computer analyses for two embankments, are listed in the remarks column of Table IV and indicate factors of safety in excess of those calculated by other means. It is a matter of regret that the embankment failures examined on the Bristol Channel alluvium have no reliable pore pressure records, to permit a check on the accuracy of the various methods of analysis considered.

The Slip Circle/1 analysis is considered to be more realistic
than the other methods, based on the higher degree of accuracy available in modelling the embankment and soil profiles.

6.4 GENERAL STABILITY CHARTS

It was necessary during the construction of embankments on the motorway to provide the site staff with a rapid means of interpreting the information obtained from the monitoring instruments. A rapid and general form of effective stress analysis was required to use the results available from the piezometers as a continuous check on stability. Observations of the deformation of embankments under controlled loading also gave a continuous indication of stability. The system of control had also to be capable of predicting the effect of placing further fill at any stage of loading.

Design charts were prepared by J M McKenna, based on the stability analyses of two embankments with varying average degrees of consolidation under the centre line of the embankments. The stability analyses were computed using the Little and Price (1958) computer programme. The design charts are shown in Fig 33 and consider the analysis of an embankment in terms of the pore pressure dissipation under the centre of the embankment, expressed as a percentage of the embankment load (ie \((1 - ru)\)). It was stated (McKenna 1973) that the pore pressures developed under the shoulder of an embankment were more critical to the stability of the embankment. This was
Assumptions:
1. Maximum pore water pressure at 7.3m below original ground level.
2. Minimum Factor of Safety at 4.9m below original ground level.

GENERAL STABILITY DESIGN CHARTS, after McKenna 1972  FIG. 33
also found from work done by Parry (1954) on stress redistribution under a model granular embankment. To account for this two lateral pore pressure distributions were analysed. The two distribution curves A and B shown in Fig 34 were derived from the measured pore pressures under the two embankments analysed. Design charts were prepared for both distribution curves.

The slip circle analyses considered two stages of embankment loading namely 100 and 150 kN/m\(^3\) and varying degrees of consolidation under the centre of the embankment. The foundation soil was assumed as a soft clay with effective strength parameters
\[ c' = 7.66 \text{ kN/m}^2 \]
\[ \phi' = 28^\circ \] in the upper dessicated crust and
\[ c' = 0 \]
\[ \phi' = 28^\circ \] in the lower soft clays. The maximum pore water pressure occurred at a point 7.3m below ground level, but the minimum factor of safety was found for slip circles tangential to a line 4.9m below ground level.

The factor of safety of an embankment was calculated in the following manner, after McKenna 1973:-

1. On a cross section of the embankment, the actual excess pore-water pressures were plotted, and the percentage consolidation under the centre line to a depth of 7.3m was calculated.
Pore water pressure distribution curves

Under embankment

Outside embankment

DISTANCE FROM CENTRE LINE - m

DISTANCE EITHER SIDE OF TOE OF EMBANKMENT - m
2. Choosing the pore pressure distribution curve closest to the field conditions, the pore water pressures under the shoulder at each piezometer position were calculated and plotted on the cross section, and visually compared with the actual values. If the comparison was poor, then by a trial and error process the percentage consolidation on the centre line was altered, until the calculated values under the shoulder and the actual values agreed.

3. Knowing the embankment loading and the actual or assumed percentage consolidation on the centre line, the stability chart corresponding to the chosen pore water distribution curve was used to determine the factor of safety of the embankment.

4. The effect of placing further fill was investigated by extrapolating the graphs of pore water pressure against embankment load. From the extrapolated values, the factor of safety was recalculated.

The factor of safety was allowed to fall to 1.2 before embanking was stopped. The excess pore water pressures were then allowed to dissipate before any further layers of fill were placed. In the case of the Westonzoyland Road embankment the rate of pore pressure dissipation was not fast enough for the embankment to be completed within the available contract period.
EXCESS PORE PRESSURES VS AVERAGE DEGREE OF CONSOLIDATION
(after McKenna 1971)

FIG. 35
and required wide berms to be constructed to enable the design height to be achieved.

Appendix VIII(C) sets out a typical calculation for the determination of the factor of safety of a side road embankment.

The use of the design charts, provided a rapid means of assessing the stability of an embankment in terms of effective stress, which was essential to control the placing of fill. Subsequent computer analysis of the results indicates that the factor of safety indicated from the design charts is too low. This is believed to result from the incorrect assumption of pore water pressure distribution.

A factor which also has a marked influence on the calculated stability, is the selection of a value of $c'$. In the analyses a value of $c' = 7.66 \text{ kN/m}^2$ has been used for the upper clay crust and $c' = 0$ for the softer clays. Parry (1971) indicated that using a value of $c' = 1.9 \text{ kN/m}^2$ gave a factor of safety in excess of unity on a failed embankment. In view of this observation and the inherent difficulties in measuring small values of $c'$ with any great accuracy in the laboratory, he suggests that $c' = 0$ should be adopted in stability analyses.
CHAPTER 7
SETTLEMENT CALCULATIONS

7.1 INTRODUCTION

The embankments constructed across the Somerset Levels were to increase the loading on the soft clay subsoil to a level that it had not experienced before.

In order to predict the amount of settlement of embankments constructed on the alluvium, it was necessary to measure accurately the degree of over-consolidation of the clay. The determination of $P_{c'}$, the preconsolidation pressure, is partly a function of the test procedure adopted in the laboratory and the effects of sample disturbance. Details of the test procedure are described later in this Chapter.

The different factors which can give rise to the preconsolidation pressure being greater than the present effective overburden pressure are summarised by Bjerrum (1967, 1972) and Simons (1974). These factors can either act independently or together and are:

(a) removal of overburden
(b) fluctuations in the ground water table
(c) cold welding of mineral contact points between particles
(d) exchange of cations
(e) precipitation of cementing agents
(f) geochemical processes due to weathering

(g) delayed consolidation

Of these possibilities the first two were considered unlikely as the results of a site investigation for another section of the motorway showed virtually no over-consolidation. The possibilities listed as c, d, e and f could be neither proved nor disproved and they may be the reason for the very variable vane test results, which could however be due to the organic particles in the clay.

The hypothesis of delayed consolidation was advanced by Bjerrum (1967) to explain the settlement of buildings in the Norwegian town of Drammen, 40km south west of Oslo.

The results obtained from the site investigations for the motorway showed sufficient agreement with the Norwegian results for it to be considered that the Somerset clay would behave in a similar manner. Calculations based on Bjerrum's work are set out in this Chapter.

There is a degree of disparity between the settlements predicted by Soil Mechanics Ltd (1970) and the writer. These settlements together with the actual settlements at the end of the one year surcharge period are shown in Fig 42.
7.2 **SETTLEMENT CALCULATIONS**

The settlement calculations have been based on the results of oedometer tests on 75mm diameter samples, obtained from 100mm diameter piston samples. As one of the objects of the test was to determine the pre-consolidation pressure, an incremental loading factor of 1.5 was used in some of the tests, the remainder were done using the standard factor of 2.0 (Casagrande et al 1965).

Fig 36 shows a typical plot of voids ratio against logarithm of vertical effective pressure \( e - \log p \) from which the pre-consolidation pressure \( P_c' \) can be estimated using the Casagrande (1936) graphical construction. The \( e - \log p \) curve was re-constructed to field conditions using the Schmertmann (1953) method.

In Fig 8 the preconsolidation pressures for the Somerset clay are plotted against depth, after Soil Mechanics Ltd (1970) and in the lower half of the figure the overconsolidation ratio, the preconsolidation pressure \( P_c' \) divided by the effective overburden pressure \( P_o' \), is plotted against the water content. It has been assumed from these results that the overconsolidation ratio is equal to 1.6, which is the same value as determined for the Drammen clay.

Bjerrum derived what was believed to represent a unique relationship between void ratio, overburden pressure and time. This relationship is shown in Fig 37(b) and means that for any given value of
Description: Firm grey friable very silty CLAY with vegetation throughout

Depth below surface 4.57 m

Moisture content 47%

Initial void ratio 1.23

Degree of saturation 100%

Specific gravity 2.62

Bulk density 16.97 kN/m³

Dry density 11.63 kN/m³

EQUILIBRIUM VOIDS RATIO VS LOGARITHM OF THE VERTICAL PRESSURE - FIG. 38
FIG. 37

overburden pressure and void ratio there corresponds an equivalent time of sustained loading and a certain rate of delayed consolidation, independent of the way in which the clay has reached these values. This relationship gives a load-deformation model incorporating secondary settlement.

Bjerrum considered compression in terms of "instant" and "delayed" compression. These terms are contrary to the conventional concepts of primary and secondary compression, which relate to compression before and after dissipation of excess pore pressures. Fig 38 shows the relationship between the two definitions for a clay layer loaded with a suddenly applied uniformly-distributed pressure. The term "instant" relates to a condition, where the applied pressure is transferred instantaneously to the clay structure as an effective pressure and achieves an immediate reduction in void ratio in the clay structure supporting the overburden pressure. He expressed this relationship between "instant" and "delayed" compression in the following equations:

\[
\varepsilon_{i} = \frac{Cc}{1 + eo \log \frac{Pc + \Delta p - (Pc - Po)}{Pc}} \tag{1}
\]

\[
\varepsilon_{t} = \frac{Cc}{1 + eo \log \frac{Po + \Delta p}{Po}} \tag{2}
\]

where the delayed compression is derived from equation (2) minus equation (1). Equation (2) is the normal expression developed in

\*Secondary settlement being the term used to describe the settlement which takes place after the excess pore pressures have dissipated, i.e. settlement at constant effective stress.
DEFINITION OF INSTANT AND DELAYED COMPRESSION COMPARED WITH PRIMARY AND SECONDARY COMPRESSION—after Hjerrum (1967)

FIG. 38
Terzaghi's theory of one-dimensional consolidation for vertical strain due to consolidation and is derived from the oedometer e - log p curve.

Total compression represents the compression which can be expected, at a time after the load has been applied equal to the time elapsed since the clay was deposited. From the family of curves derived for the Drammen clay the percentage of delayed compression can be expressed against time as in Fig 37(a). The Somerset clays related to this curve show that they have undergone 95% delayed compression in the time since deposition.

Bjerrum's model for explaining the phenomenon of delayed consolidation is based on the results of oedometer tests conducted over a period of 30 days and the long-term observations of settlement on buildings in the Drammen area. The system of lines or curves derived in this way to represent the 30, 300 and 3000 year relationship for equilibrium void ratio and different values of effective overburden pressure, may or may not be correct.

Simons (1974) refers to the considerable importance in distinguishing between preconsolidation pressure due to (a) overconsolidation as a result of the removal of overburden pressure, or groundwater level fluctuations or chemical and weathering effects and (b) delayed consolidation. In the first case there is some evidence to suggest that the clay can be loaded to near the preconsolidation
pressure with small resulting settlements. The section of the
A38 Trunk Road referred to in Chapter I was constructed on the
Somerset clay, on a 1.3m embankment. There has been no
measurable settlement on this section of road, since construc-
tion in 1966, due to the additional loading being less than
$P_{c'} - P_{o'}$.

If the preconsolidation pressure is due to delayed compression
then from Bjerrum (1967) significant settlements can be expected
when the applied pressure exceeds 50% of $P_{c'} - P_{o'}$. Case
records reported by Simons (1974) of work done in Sweden by
Nordin and Svensson (1974) and in Norway by Andersen and
Primann Clausen (1974), appear to conflict with Bjerrum’s findings.
In Sweden the measured settlements under buildings in Lidköping
and Vanersborg were of the order of 9mm and 16mm and in the
first case the value of $\frac{\Delta p}{P_{c'} - P_{o'}}$ was about 0.5, while in the
second case the increase in stress due to the building was
nearly equal to $P_{c'} - P_{o'}$. In Norway settlement records for a
structure indicate marked delayed compression while the results
from oedometer tests indicate $P_{c'}$ approximately equal to $P_{o'}$.

In view of the evidence from these case records it is apparent
that laboratory studies alone will not allow accurate settlement
predictions to be made. Indeed from Simons (1974) it is apparent
that few clays exhibit a preconsolidation pressure due to
delayed consolidation.

$\frac{\Delta p}{P_{c'} - P_{o'}}$

*The M5 motorway embankment indicated a value of $P_{c'} - P_{o'}$ of
approximately 140% and large settlements were therefore expected.*
The design of the embankments on the Somerset alluvium was based on Bjerrum's concept of delayed compression and the embankments were designed for settlements at the end of fifty years. The surcharge design was related to achieving the equivalent settlement in one year, less an allowance of 150 mm for post-construction settlement.

In Fig 37(b) the overconsolidation ratio $\frac{P_{c'}}{P_{o'}}$ can be seen to decrease with increase in time from the application of the additional load. Until at 3000 years the ratio $\frac{P_{c'}}{P_{o'}} = 1$. This is shown in the graph of the overconsolidation ratio against time in Fig 37(c). Therefore the design settlements at one year and fifty years from the date of construction can be calculated from equation (1) where $P_{c'}$ equals the overconsolidation ratio at one year and 50 years multiplied by the initial effective overburden pressure $P_{o'}$.

The calculations for the settlement of a typical section of the motorway are detailed below.

The distribution of vertical stress beneath an embankment has been calculated from the Influence Values established by Fadum (1956). The embankment was 50m wide and the vertical pressure remained reasonably constant to a depth of 8m under the centre line.

*For calculation purposes the construction of the embankments has been assumed as an instantaneous loading halfway through the construction period.*
The stress diagrams and borehole log are shown in Fig 39 and indicate a seven-layered structure. The settlement calculations have assumed two-way drainage throughout the clay strata.

(a) **SETTLEMENT CALCULATION BY CONVENTIONAL METHOD**

Consider an embankment loading of 3m of heavy fill ($\gamma = 21.2\text{kN/m}^3$). Then, Settlement ($S$) is expressed by Terzaghi's one dimensional consolidation theory as:

$$S = \frac{Cc}{1 + eo} \frac{2H}{\log 10} \log \frac{Po' + \Delta p}{Po'}$$

where $Cc$ = compression index of the clay  
$eo$ = initial void ratio  
$H$ = thickness of the clay layer  
$Po'$ = existing effective overburden pressure  
$\Delta p$ = applied effective pressure

The relationship between compression ratio $\frac{Cc}{1 + eo}$ and moisture content has been established for the Somerset clays by Soil Mechanics Ltd (1970) and is shown in Fig 40.

For layer (1)  
$$S = 0.105 \times 2.13 \times \log \frac{9.01 + 64.7}{9.01}$$  
$$= 0.204m$$

(2)  
$$S = 0.158 \times 2.13 \times \log \frac{25.11 + 64.7}{25.11}$$  
$$= 0.186m$$
Vertical stress \( \sigma_z = (1.1 \times q) \times q \)

after Fadum (1950).
COMPRESSION RATIO VS WATER CONTENT

FIG. 40
For layer (3) \[ S = 0.440 \times 1.067 \times \log \frac{32.51 + 64.7}{32.51} \]
\[ = 0.223 \text{m} \]

(4) \[ S = 0.250 \times 5.639 \times \log \frac{51.61 + (64.7 \times 0.976)}{51.61} \]
\[ = 0.489 \text{m} \]

(5) \[ S = 0.186 \times 1.829 \times \log \frac{76.40 + (64.7 \times 0.94)}{76.40} \]
\[ = 0.087 \text{m} \]

(6) \[ S = 0.116 \times 2.438 \times \log \frac{89.70 + (64.7 \times 0.914)}{89.70} \]
\[ = 0.062 \text{m} \]

\[ S = 1.251 \text{m} \]

Then from the equation \[ t = \frac{TvH^2}{Cv} \]
where \( t \) = time

\( T_v \) = pure number called the 'time factor'

\( C_v \) = coefficient of consolidation*

Then in one year

\[ T_v = \frac{t \times C_v}{H^2} \]
\[ = \frac{1 \times 13.935}{6.4^2} = 0.34 \]

*From a study of the settlements of the East Brent and Clevedon trial banks in the year following the end of construction, Soil Mechanics Ltd (1970) arrived at a value of \( C_v \) for the clay and peat beds of 13.935m²/year.
From Taylors (1948), curves of Time factor against Degree of consolidation (U%) then

$$U = 65\%$$

For layers (1) - (5) one year settlement $$S = 0.65 \times 1.189 = 0.773m$$

Total one year settlement = $$0.773 + 0.062 = 0.835m$$

Where layer (6) is assumed to have a Cv value of 0.0.

In 50 years

$$T_v = \frac{50 \times 13.935}{6.4^2} = 17.01$$

$$\therefore \ U = 100\%$$

$$\therefore \text{50 year settlement} = 1.251m$$

(b) BJERRUM'S SETTLEMENT CALCULATIONS

Considering the same section of motorway with the same embankment loading:

From Fig 37(c) the one year settlement represents a value of the over-consolidation ratio of 1.44.

Then

$$\varepsilon_{1 \text{ year}} = \frac{Cc}{1 + eo} \times 2H \times \log_{10} \left(1 + \frac{\Delta^2 - (P_c - P_o)}{1.44 P_o} \right)$$

For layer (1)

$$\varepsilon_{1} = 0.105 \times 2.134 \times \log \frac{73.72}{12.98} = 0.169m$$

(2)

$$\varepsilon_{1} = 0.158 \times 2.134 \times \log \frac{89.81}{36.16} = 0.133$$
For layer (3) $\varepsilon_1 = 0.44 \times 1.067 \times \log \frac{97.21}{46.89}
= 0.149$

(4) $\varepsilon_1 = 0.25 \times 5.639 \times \log \frac{114.70}{74.36}
= 0.265$ m

(5) $\varepsilon_1 = 0.186 \times 1.829 \times \log \frac{137.24}{110.09}
= 0.033$

(6) $\varepsilon_1 = 0.116 \times 2.438 \times \log \frac{148.93}{129.08}
= 0.018$

$\varepsilon_1 = 0.767$ m

The fifty year settlement can be calculated in the same manner using a $P_e/P_0$ ratio of 1.14 and gives $\varepsilon_{50} = 1.077$ m. These results compare reasonably well with the values of 0.835 m and 1.251 m obtained by the conventional method and therefore imply that most if not all of the settlement is due to delayed compression.

(c) SOIL MECHANICS SETTLEMENT CALCULATIONS

The settlement calculations produced by Soil Mechanics Ltd (1970) are shown in graphic form in Fig 41 and referred to in more detail under surcharge calculations later in this Chapter.
FIG. 41

SETTLEMENT AND SURCHARGE CALCULATIONS, after Soil Mechanics Ltd (1970)
The values of settlement indicated from Fig 41 for the 3m embankment are:

- 1 year: 0.494m
- 50 year: 0.930m

In order to compare the accuracies of the various methods outlined, the actual settlements one year after construction, have been plotted in Fig 42 for a section of the motorway. The estimated settlements from Soil Mechanics Ltd (1970), and those calculated at various increments of loading by the other methods, are overplotted.

A comparison of the results indicates that for the lower surcharged embankments (i.e. < 4.5 metres) the field rate of consolidation is less than that assumed in the design calculations and for embankments over 4.5 metres it is greater than assumed. From an analysis of the settlement performance records for a section of the motorway embankment at 3.6 metres height, (62 kN/m² loading) the field coefficient of consolidation (Cv) is approximately 8 m²/year. Whilst similar analyses of sections of embankment at 4.5 and 5.0 metres height indicates field Cv's of 20 m²/year, and 40 m²/year.

The variation in the rate of consolidation for the section of motorway embankment considered, is a function of a limited number of consolidation test on small diameter (75mm) samples, which may
SETTLEMENT AND SUBCHARRE DRAWING, after McKenna [1973]

FIG. 42
not reflect the true permeability of the clay strata and the inherent uncertainties in the assumption used in the settlement calculations. Some of the uncertainties involved in the calculations are:-

(a) The length of the drainage path in areas where the soft clay extends to bedrock, particularly in the Withy Road area from Chainage 4270 - 4570 where the soft laminated clays below 11.3 metres were assumed to act as a free drainage layer.

(b) The accurate determination of the preconsolidation pressure from laboratory tests.

(c) The use of an overconsolidation ratio \((\frac{P_c}{P_o})\) of 1.6, which is uniform with depth.

(d) The application of a limited number of oedometer test results to a long section of embankment where the stratigraphy of the alluvium varies in a relatively short distance, see Fig 2.

(e) The assumption that the various soil layers in the section considered, drain in both directions.

(f) The limitations imposed in using a one-dimensional model to calculate settlements from laboratory tests.
7.3 **SURCHARGE CALCULATIONS**

The studies of settlement on the trial embankments at East Brent and Clevedon, (Soil Mechanics Ltd 1968, 1969) one year after construction, indicated a value of the coefficient of consolidation of 13.94 m²/year. On the assumption that the conditions pertaining to the trial embankment sites could be considered as applying generally for the remainder of the motorway on the Somerset Levels, then surcharging embankments appeared to be a feasible technique.

The surcharge was designed to limit the post construction settlement on the motorway to a value of 150 mm in the 50 years following construction. This was regarded as an acceptable level of settlement, particularly at underbridge sites, where maintenance due to differential settlement was a problem. The maximum time available for surcharging was one year.

The detailed results published by Bjerrum (1967) were used by Soil Mechanics Ltd (1970) to establish the basic parameters shown in Fig 37 and referred to previously in this Chapter. The pre-consolidation pressure \( P_c' \) has been assumed as 1.6 times the effective overburden pressure \( P_o' \) and the effect of delayed compression has been taken into account by varying the over-consolidation ratio \( \frac{P_c'}{P_o'} \) with time.

The surcharge calculations represent an extension of the settlement...
calculations referred to previously in this Chapter. From Fig 41 it can be shown that the surcharge design can be interpreted directly from the load, settlement curves relating to time of loading.

In Fig 41(a) the nominal height of fill lines have been determined in relation to the settlement calculations. An embankment constructed in heavy fill (\( y = 21.2 \text{ kN/m}^3 \)) to a height of 4.21m will settle to 3.0m in fifty years (i.e. 1.21m of settlement). In order to calculate the amount of surcharge required to achieve this amount of settlement in one year, less the post construction allowance of 150mm, it is sufficient to read from the one year settlement curve the load which will produce this settlement (1.06m). A loading of 6.71m of fill (\( y = 21.2 \text{ kN/m}^2 \)) will produce a settlement of 1.06m in one year.

In order to construct an embankment for 50 year settlement, less post construction settlement, it is therefore necessary to build it 6.71m high initially and leave it in place for one year.

The relationship between nominal height of fill and surcharge loading has been expressed in a more direct form in Fig 41(b) and (c) and relates to the soil profile assumed for this section of motorway shown adjacent to Fig 41(b).

*A computer programme was written to produce these curves by the Somerset Sub Unit of the South Western Road Construction Unit and provided a print out of levels on the motorway embankment prior to and immediately after removal of surcharge loading.*
The surcharge calculations assumed uniform dissipation of pore pressure with depth, which is not valid, but was felt to be an acceptable approximation in view of the inherent uncertainties in the assumptions used in the settlement calculations. The actual distribution of pore pressure with depth was taken into account when analysing the field performance records, before surcharge loading was removed.

McKenna (1973) emphasised the importance of accurately calculating the settlement at the end of the one year settlement period. Where the observed settlement was in excess of the calculated value the surcharge loading was reduced by the amount of the excess settlement. These areas had to be identified early in the settlement period in order to ensure that additional surcharge could be placed to maintain the preload.

The additional settlement could be due to more rapid consolidation than that assumed in design and this factor had to be checked against the settlement and pore pressure records, as shown in Figs 14 and 15.

Bjerrum (1972) presented the diagram shown in Fig 43 to illustrate the principle of the design of a preload to reduce the rate of secondary compression in clays. It can be seen from this figure that two important points emerge in the design of a preload:
Principle of design of surcharge, after Bjerrum (1872 - fig 12)

Fig. 43
(i) the surcharge loading should be sufficiently large to effect the desired reduction in the rate of secondary compression for the period of loading,

(ii) the loading should be left on for a sufficient period to reduce the void ratio to the desired rate of secondary compression.

On sections of the motorway the surcharge loading was increased to allow for settlement in excess of the designed value and in other areas the loading was retained beyond the original surcharge period.

The full effects of the designed surcharge will not be known for many years, but from the post construction records it would appear that the surcharge has in most cases eliminated working life primary consolidation and has gone a long way towards reducing secondary consolidation.

The observation of settlement and pore pressure dissipation on the embankments is being continued in order to enable the long term behaviour of the embankments to be clarified.
CHAPTER 8

POSSIBLE GROUND STRENGTHENING WORKS

8.1 INTRODUCTION

Techniques of ensuring the stability of embankments on soft clay were examined during the trial embankment construction at East Brent and Clevedon. Other methods had been studied by reference to work in Norway and Sweden and the eventual solution, that of lightweight fill and pre-loading, was adopted as being the most reliable and economically the most suitable.

This Chapter examines the alternative methods of improving the foundation for embankments on soft ground that were considered and suggests some of the reasons why the different methods were not adopted.

The use of light-weight fill, namely PFA, had been allowed for in the design of the higher embankments and on a section of the motorway embankment, and has not therefore been considered as an alternative in this Chapter.

There are a number of other methods of improving the foundation of the embankments which have not been discussed, such as Electro-osmotic stabilisation, the Kjellman filter drain installations and the use of Dynamic Consolidation. These techniques have only recently been
developed commercially in this country and for this reason they were not considered in the design of the embankments on the motorway.

8.2 SAND DRAINS

The objective of sand drains is to accelerate the drainage in low permeability soils in order to obtain the required consolidation within the available period of time.

The design, construction and behaviour of sand drains was reviewed by Johnson (1970) and Bjerrum (1972) and emphasised the importance in evaluating correctly the significant characteristics of the subsoil. There are two important properties of the subsoil which influence the efficiency of sand drains, these are permeability and compressibility.

Bjerrum related the efficiency of a sand drain installation to a factor \( \eta \) where

\[
\eta = \frac{c}{\text{c}}
\]

and 

\( c = \) settlement at the end of primary consolidation

\( \text{c} = \) total settlement, i.e. primary plus secondary settlement

The effect of a sand drain is to accelerate primary consolidation, but will have no influence on secondary settlements. The efficiency
of an installation therefore depends on the amount of settlement which can be accelerated by the sand drains compared with the total settlements.

The analysis of the failure of the East Brent trial embankment suggested that there had been a high transfer of pore water pressure from under the centre of the bank to under the toes. One of the purposes of the Clevedon trial embankments was to check the possibility of the high pore pressure transfer and to examine the effectiveness of sand drains in preventing it.

The sand drains were installed in a single row 5 metres in from the toe, and consisted of 305mm diameter holes at 1.83m centres extending to a depth of 7.9m. The holes for the sand drains were formed by a hollow stemmed flight auger, with the intention that the hole should be filled with sand through the hollow stem as the auger was being withdrawn. Difficulties were encountered in placing the sand and most of the holes were filled after the auger had been removed. The piezometers located under the bank and beyond the toe indicated no significant pore water pressure transfer and the settlement records showed no appreciable increase in the rate of settlement in the sand drained area. This is somewhat in conflict with Bjerrum's findings as the efficiency factor for this site is of the order of 0.60, which would indicate that sand drains could be effective.

Sand drains were ultimately considered unsuitable for the motorway embankments for the following reasons:
i. It was considered generally that the disturbance caused by installation of the sand drain by driving and the effect of smear, offset the beneficial effects of the drain.

ii. Sand drains were considered unnecessary in the peat and the laminated clays; and this is endorsed by Bjerrum (1972).

iii. Experiences in this country, namely on the Kings Lynn trial embankment (Wilkes 1972) and on the Thames Estuary (Lewis 1963) indicated that from an engineering aspect, sand drains are not an economic proposition.

iv. It is now apparent by reference to Bjerrum (1972) that the low motorway embankments would not have benefited from sand drains as the applied load would only have been marginally in excess of the preconsolidation pressure.

8.3 VIBRO-FLOATATION

Vibro-floatation refers to a technique developed by Messrs Cementation for the formation of stone piles. Prior to the construction of the East Brent trial embankment an area 30.48m long and 70.2m wide was treated at one end of the bank using 900mm diameter piles approximately 11.3m long and formed at 2.29m by 2.44m centres.

The piles were installed by vibrating into the soil a mass of
stones to form a pile, the base of which penetrated the fine silty sands. The object of the stone piles being to reduce the amount of settlement occurring in the soft clay.

The report on the failure of the embankment, Soil Mechanics Ltd (1968) noted that the stone piles at one end of the bank seem to have had no effect on the settlement performance of the bank. The treated area and the centre section of the bank having settled 1.37m at the time of the report. In addition it appeared that the effects of the slip extended into the piled area.

As a result of the experiences at the East Brent trial embankment the vibro-floating technique was not adopted for the motorway embankments.

8.4 RELIEF PILES

In European countries where there are large areas of extremely soft clay, relief piles are used to a considerable extent to solve the problem of settlements of embankments next to bridge abutments and on the flat areas where there are no problems of bearing capacity. The piles are generally wooden and may be capped with precast concrete pile caps or simply a layer of sand to transfer the load to the piles. Alternatively no capping is used and the friction between pile and clay is relied on for transfer of load to the pile.
Eide (1968) reviewed the use of driven timber relief piles for the Nakhon - Sawan highway in Bangkok. The piles are inclined towards the edge of the embankment, as shown in Fig 44, to reduce horizontal movements and shear deformations in the clay. The use of relief piles adjacent to a bridge abutment is also shown in Fig 44, and shows that the length of the piles is gradually reduced with the distance from the abutment. In this way a gradual transition can be achieved in the settlement profile.

The use of relief piles was not considered economically viable in view of the large depths of compressible material in some areas.

8.5 **BERMS**

A recognised technique for ensuring the stability of a road embankment on weak foundations is to construct supporting embankments on the sides of the embankment. These are referred to as berms and serve to reduce the settlements resulting from shear deformation in the clay.

The design of the motorway embankments considered the use of berms, as an emergency measure, should incipient failure develop.

The use of berms does however increase the stresses on the deeper strata causing an increase in the consolidation settlements. This factor, combined with the high increase in land acquisition* for

*Land acquisition for the motorway amounted to 6.0 hectares per kilometre and the provision of berms would probably have involved a 25% increase in this area.
RELIEF PILES TO REDUCE HORIZONTAL MOVEMENTS AND SHEAR DEFORMATIONS IN A CLAY, after Eide (1968) FIG. 44
the provision of berms effectively excluded their use on the motorway.

8.6 PRELOADING

The design of a preload to improve the foundation soils can be done with little certainty as to the effect of the load in eliminating primary consolidation, during the period prior to construction of the structure; in this case a road pavement. It was not anticipated in the design of the preload for the motorway embankment that there would be a significant reduction in secondary settlement. In fact the design incorporated what was considered an acceptable level of post construction settlement namely 150mm.

The use of a preload was adopted and a period of 12 months was allowed in the Contract for the loading to remain in position. The amount of settlement and the rate of settlement had to be accurately monitored in order that the areas of high settlement could be identified and related to the design rate of consolidation. Those areas where the settlement was due to more rapid consolidation than the design, had additional surcharge placed to maintain the preload.

The design of the surcharge and the settlement calculations have been discussed in the previous Chapter and it is sufficient to say that the use of surcharge loading throughout the motorway from Clevedon to Huntworth proved very successful in reducing the amount
of post construction settlement to an acceptable level.

The economic advantages in using the preload technique were obvious, when it was decided to re-use the surcharge material on successive motorway contracts. The programme of work was thereby arranged to permit the re-use of this material from Clevedon to Huntworth, where it was eventually disposed of in a landscaped cutting.

There would appear to be a need for further research studies of the performance of sand drains to accelerate construction and reduce post construction settlements. There are few examples available in this country, which prove the results achieved from the use of sand drains, from a direct comparison between the rates of settlement observed in a sand drained and a non sand drained area.

A lot of work has been done in Europe and in particular Sweden to study the effect of sand drains and the conclusions reached indicate that they have a very beneficial effect.

The method of constructing the sand drains is one aspect which requires careful attention, in order to ensure that the level of disturbance in placing does not outweigh the benefits to be achieved from the use of the drain. The remoulding of the sensitive clays due to driving of the drains needs to be investigated further in order to ensure that this does not result in an
increase in the primary compression or has any detrimental
effects on the primary settlement rates.

The use of the Kjellman fitter drains has also to be proved
beneficial in accelerating construction and reducing post
construction settlement in this country.
CHAPTER 9

CONCLUSIONS

The design and construction of embankments on alluvial clays presents considerable problems due to the low shear strength and high compressibility of the soil.

Clearly the determination of the undrained shear strength of the soil is an important factor in the design of embankments and extreme care should be exercised in obtaining, handling and testing samples in order to minimise the factors which can influence the measured shear strength. Some of the factors which are known to influence significantly the measurement of the undrained shear strength of the clay have been referred to in Chapter 5 and are summarised by Parry (1971) and Bjerrum (1972) and include:

(i) Sample disturbance
(ii) Rate of strain
(iii) Anisotropy
(iv) Structural features
(v) Stress redistribution
(vi) Progressive failure

It is clear from Chapter 5.3 and Fig 29 that the disparity between the undrained shear strengths measured in the unconfined compression tests and the in-situ Geonor vane tests, partly reflect the influence of these factors, on the Somerset alluvium. The average minimum unconfined compression test results were approximately 50% less than
the average minimum in-situ Geonor vane test results. Applying
the correction factor $\mu$ from Fig 28, as suggested by Bjerrum (1972),
to the undrained shear strength measured by the vane, reduces the
disparity by 33% as referred to in Chapter 6. The results of the
stability analyses listed in Table IV show a better overall agree­
ment between the analyses using the corrected vane shear strengths
and the effective stress analyses using measured pore pressures.
Parry (1971) suggests that effective stress analyses provide a better
agreement with field conditions than total stress analyses and
from the observations of ground movement on the motorway and side
road embankments, it is apparent that at no time was a condition
of imminent failure reached. This would appear, from the results
listed in Table IV, to endorse the effective stress analysis as a
more accurate assessment of field conditions.

The effect of consolidation of the clay during construction of the
embankments may well account for the fair agreement between theory
and practice.

There is an apparent need for more research into the relationship
between the various undrained shear strength data and the analytical
model in field stability problems.

It is evident from Bjerrum (1972) that different tests (Fig 45) to
measure the undrained shear strength of a clay produce different
results. The shear test to be used in an analysis of embankment
stability must therefore be selected to match the distortion
expected in the field. Alternatively the results from the various
MODES OF FAILURE ALONG A SLIP SURFACE, after Bjerrum (1972)

FIG. 45
tests must be corrected to produce an average value of undrained shear strength around the failure surface. Bjerrum (1972) suggested that the field value of shear strength could be reliably computed by applying correction factors to the undrained shear strength as measured by the vane. The field value could then be expressed as:

$$Su(\text{field}) = Su(\text{vane}) \mu_R \mu_A$$

where $\mu_R$ is a factor correcting for the rate effect which depends on the plasticity index of the clay.

$\mu_A$ is a factor correcting for the anisotropy of the clay and it will vary along a sliding surface depending on its inclination and the plasticity of the clay.

Bjerrum (1972) refers to the results of comprehensive studies, at various laboratories, from which it has been found that anisotropy can be measured reliably from undrained tests on undisturbed samples, provided they are consolidated anisotropically. The procedure involves testing samples in compression and extension in a triaxial cell and a direct simple shear test, representing the three directions of failure shown in Fig 45. Typical results of undrained shear strength from these tests are shown in Table V and are expressed as a ratio of the effective overburden pressure ($P_o$).

The effect of anisotropy can be clearly seen from Table V and is
<table>
<thead>
<tr>
<th>Type of Soil</th>
<th>Index properties</th>
<th>Triaxial test $\tau / P_o$</th>
<th>Simple shear test $\tau / P_o$</th>
<th>Vane tests $s_v / P_o$</th>
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<td></td>
<td>$w$</td>
<td>$w_d$</td>
<td>$w_p$</td>
<td>Ip</td>
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<td>Drammen Lean clay</td>
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<td>11</td>
</tr>
</tbody>
</table>

Comparison between the results of compression and extension tests, direct simple shear tests and in-situ vane tests on soft clay (after Bjerrum, 1972)
greater in lean clays, reducing as the clays become more plastic.

Menzies (1974) introduces the principle of similitude in assessing the applicability of shear test data to the analytical model and the field prototype. This relationship, as outlined by Parry (1971) and Bjerrum (1972), is regarded as an important factor and probably accounts for the disparity in the results obtained from the unconfined compression test and in-situ vane tests on the Somerset alluvium. Neither of these tests, as detailed in Appendix I, can be regarded as simulating the field prototype.

Bjerrum (1972) suggests, however, that vane shear strengths corrected in relation to Fig 28 appear to nearly equal the average shear strength observed in the three types of laboratory test shown in Fig 45 and listed in Table V. This probably explains why it is possible to use the corrected vane shear strengths to compute the stability of embankments.

The influence of progressive failure is not considered in the conventional stability analysis. The effect of considering the peak shear strength to act on all elements simultaneously around the failure surface overestimates the factor of safety. It is also clearly inappropriate to consider the residual shear strength to act on all elements on the slip surface as this will underestimate the factor of safety and implies an established shear surface. The use of the residual factor (Skempton, 1964) goes
some of the way to compensating for the effect of progressive failure.

In the design and construction of embankments on alluvial soils it is important to establish from the site investigation the detailed stratigraphy of the alluvium, as shown in photographs A, B, C and D. The presence of rootholes, rootlets, fissures and laminations in the clay has a marked effect on the dissipation of excess pore pressures (Rowe, 1972). The examination of the continuous piston samples after they had been split open and allowed to air-dry on polythene trays enabled the structure of the alluvial deposits to be accurately determined. In the Withy Road area shown in Fig 1 the clay extended to bedrock, providing a thickness of clay of approximately 25 metres. The lower half consisted of silty laminated clays and it was uncertain whether the rate of consolidation in this area would relate to the calculations for the remainder of the motorway. From Fig 14 it can be seen that the rate of consolidation north of the River Huntspill, adjacent to Withy Road, was slower than the area south of the River Huntspill (Fig 15) where silty fine sands replaced the laminated clays. The rate of consolidation in the Withy Road area does suggest however that the laminations in the clay provided the lower drainage paths required for the overlying silty clays.

In order to achieve a post construction settlement of 150mm required in the design of the motorway embankments it was necessary
to preload the clay soil. The predicted and observed values of settlement shown in Fig 42, for the preloaded embankments one year after construction, show a reasonable agreement for an embankment height of 4.5 metres. Above and below this height the observed values of settlement differ from the predicted, as shown in Fig 42. It is evident from this comparison that the rate of consolidation varies for different sections of the motorway. This was anticipated at the design stage and an allowance of a further 150 to 300 mm in excess of the theoretical post construction settlement was recommended. Where the actual settlement was in excess of the calculated value the surcharge loading was increased to maintain the preload. These areas of high settlement were identified, from the monitoring, at an early stage so that the additional surcharge could achieve the maximum effect within the twelve months surcharging period.

It is interesting to observe from Chapter 7 that the results of the conventional settlement analyses shown in Fig 42, show a reasonable agreement with the results obtained from Bjerrum's hypothesis of delayed consolidation, implying that most if not all of the settlement is due to delayed compression and that this effect has been adequately modelled in the oedometer test.

There is clearly scope for more research work on the effects of sample disturbance and sample size on the measurement of the preconsolidation pressure and the determination of the effects
of delayed consolidation. It is hoped that the post construction monitoring of the embankments will provide a useful study of the settlement time performance.

Embankment construction on the motorway and side roads was controlled by the use of instrumentation, detailed in Appendix VII.

In order to provide an accurate and rapid means of interpreting the data obtained from the piezometers and settlement gauges, a general form of effective stress stability analysis was prepared, as detailed in Chapter 6 and Appendix VIII (after McKenna, 1973). The design charts shown in Fig 33 were prepared from stability analyses carried out on a computer. The analyses were undertaken by expressing the pore pressure dissipation under the centre of the embankment as a percentage of the embankment load. Two lateral pore pressure distributions, shown in Fig 34, were analysed from pore pressures measured under two of the embankments.

The factors of safety calculated from the design charts would appear from Table IV to underestimate the stability of the embankments in relation to the effective stress analyses using actual measured pore pressures and this is believed to be due to the incorrect pore pressure distributions assumed in the general analyses.

Subsequent effective stress analyses using a terminal connection to a computer centre, has shown the value of considering a small
computer terminal, on a site such as the motorway through Somerset. Using programmes such as the Genesys Slip Circle/1 detailed in Appendix VIII the Engineer is able to input the results of the monitoring directly into an effective stress analysis and obtain the answer in a matter of minutes. The alternative is to make some simplifying assumptions of the monitoring information and produce a general impression of the stability of the embankment under consideration. The accuracy of this method is related to the assumptions made initially, usually under pressure from the Contractor whose works programme hinges on a decision as to whether to proceed with filling.

With the increasing expansion of the centres of population it is apparent that the communication systems such as roads will have to utilise the areas of land and coast line which are considered unsuitable for other uses. This will provide the engineer with more and more problems relating to the construction of roads on highly compressible or otherwise unsuitable materials. For this reason it is important to provide regional studies of this work from which the profession can benefit.

The construction of embankments over similar ground to the Somerset alluvium should, in view of the work detailed in this thesis, proceed with particular emphasis on the following points.

(1) The site investigation should be designed so that the geotechnical properties of the soil, as shown in Figs 5 and 6, can
Where practicable continuous piston sampling at 54mm or 100mm should be used to establish the structure of the soil, as referred to in Appendix I.

The current cost of a borehole to penetrate 26 metres of alluvium is approximately £500. The number of boreholes available for a survey are therefore limited. The siting of boreholes will be largely determined by structures and high embankments, but sufficient boreholes should be allowed, however, to enable the variability of the material to be determined.

(2) The size of samples should be determined in relation to the soil fabric, after Rowe (1972). For this purpose 150mm and 250mm piston samples should be considered at selected depths.

(3) Oedometer tests should be carried out at a selected rate of loading and with rest periods permitted to enable the preconsolidation pressure and coefficient of consolidation (C_v) to be accurately determined (Simons 1974). The use of the larger samples referred to in (2) above should ensure that the C_v value measured, reflects the permeability of the clay in-situ.

(4) In-situ and laboratory undrained shear strength tests should be conducted at a rate to failure related to field loading conditions (after Bjerrum 1972). Parry (1971) suggests a time to
failure of 20 minutes.

(5) Laboratory shear strength tests should be undertaken, as suggested by Bjerrum (1972), to establish the anisotropy of the clay. The effects of rate of loading and anisotropy can then be considered separately in arriving at the undrained shear strength to be used in the stability analyses.

(6) The construction of an instrumented trial embankment should be considered if the scale and cost of the project warrants it. The use of laboratory testing to determine the geotechnical properties of the soil for subsequent use in quantitative methods of analysis, presents a number of problems relating to sampling, testing and methods of analysis, referred to in Chapters 5, 6 and 7 and Bishop and Green (1973). The use of a trial embankment enables the engineer to proceed with greater certainty on the margin of safety under operating conditions and the interpretation of subsequent monitoring information.

(7) The use of vertical sand drains should be considered, perhaps in relation to (6) above, in areas where the drainage characteristics of the clay are in doubt. Studies referred to by Bjerrum (1972) of the use of sand drains in Sweden and Norway indicate that they can considerably shorten the time to consolidate the type of soils found in Somerset. The efficiency factor \( \eta \) referred to by Bjerrum (1972) indicates that for the Somerset clays, sand drains could be effective (refer Chapter 8).
(8) The use of surcharge loading, staged construction and berms referred to in this thesis should provide useful information in the design of the project. Particular emphasis should be placed on the programme of work to ensure that the quantity of monitoring information can be readily processed by the available site staff. Alternatively adequate staff should be provided to deal with an unrestricted construction programme. In either event the Contract must allow the Engineer to suspend operations, without incurring increased costs, when the monitoring information indicates it is unsafe to proceed.

(9) The design of ground strengthening works, referred to in Chapter 8, should be related to the subsequent maintenance criteria. In the case of the M5 Motoway a level of post construction settlement of 150mm was regarded as acceptable. The use of preloading has yet to be evaluated in relation to long term settlement, but it is apparent from the initial monitoring information that it has been effective in reducing primary settlement. In the case of the M5 embankments on the Somerset levels higher preloads could have been used.

Preloading may not be economic on many projects and the estimated cost of 25 mm of fill over the length of the M5 Motorway, based on 1971 prices, was £28,000.

(10) To control earthworks effectively on highly compressible
soils, then some form of instrumentation is necessary. The design of the instrumentation should be carefully considered in relation to the design of the project. Problems relating to the stability of embankments can be controlled by monitoring pore water pressure measurements, settlement and lateral ground movement. Pore water pressure measurements used with some form of general stability analysis, such as referred to in Chapter 6 and Appendix VIII, has been found to be most effective.

(11) The processing of the monitoring information presents a considerable drain on staff time. The value of the information is also limited by the inherent assumptions built into a quantitative assessment of stability. This situation could be improved by the use of a desk top computer or terminal connection to a computer centre.

The computer programmes currently available would enable an accurate assessment of stability from the monitoring information, and provide a facility for projecting this information to examine a future stage of construction.

(12) The use of lightweight fill, such as pulverised fuel ash, should be considered in the construction of embankments. The low density of the fill results in reduced problems of instability in the embankments and requires less surcharge loading to achieve the level of post construction settlement selected.
The successful construction of the M5 embankments over the Somerset Levels indicates that there are techniques available to the engineer for the monitoring of stability during construction, allowing utilisation of an area of fenland unsuitable for most other uses.
APPENDIX I
SITE INVESTIGATION

INTRODUCTION

The soil survey for the 16km section of motorway from Edithmead to Huntworth was carried out between May and August 1969 by Soil Mechanics Ltd. A secondary survey was undertaken by Geo Wimpey Ltd in 1970 to answer some of the questions raised in the previous survey.

1. BOREHOLES — During the site investigation for the motorway 96 boreholes were drilled, in 21 of which vane tests were conducted at 0.75m intervals to depth between 6 and 18m. In 28 boreholes piston samples of 54mm and 100mm diameter were taken continuously with depth.

At each bridge site two borings were made. One was used to obtain piston samples and the other for vane testing.

Thirty six of the boreholes were extended by diamond drilling in TNX (60.5mm) or T101mm* sizes and 21 holes were drilled from ground level, in the Polden Hill cut area.

The results of the survey were reported by Soil Mechanics Ltd, Report No 5333 (1970).

*TNX and T101 referring to the size of the rock cores obtained by core barrels of 60.5 and 101mm internal diameter. The larger diameter T101, was used to aid the recovery of the more shattered and fragmented rocks. It has been found that with the larger sizes the action of the drilling fluid has a less detrimental effect on the core.
ii. SAMPLING - In the firm clay forming the dessicated crust, undisturbed 100mm diameter open drive samples were taken. In the underlying clays and silts 54mm and 100mm piston samples* were taken. These were required to obtain undisturbed samples, where other techniques would fail to obtain a sample or produce an unacceptable level of disturbance.

The majority of the piston samples were extruded in the site laboratory. The samples which were not required for unconfined compression tests were extruded in 150mm lengths and split open by making a small incision along the longitudinal axis and tearing the sample apart, in order that one half could be used for moisture content determination and Atterberg limits, and the remainder laid on polythene trays to air dry before examination. This technique as outlined by Terzaghi & Peck (1967) allows the thin silty laminations and vertical root structures which influence the rate of consolidation of the soil mass to be observed. Photographs A to D show the structure of the alluvium observed in this way.

iii. TESTING - In the soft and sensitive clay soils of the Somerset levels it is difficult to obtain reasonable undisturbed

*Piston samples are obtained in a thin walled tube generally 54mm or 100mm diameter, which is closed at the lower end by means of a piston. When the sample has been pushed down to the required depth the thin walled tube is advanced ahead of the piston which remains stationary and in close contact with the top of the sample. The soil which forms the sample does not move with respect to its in-situ position until the tube is withdrawn; during this process a partial vacuum at the top of the tube assists in retaining the soil in the tube.
samples for the determination of the undrained shear strength. Where core samples are taken by driving, the disturbance often causes an appreciable loss in strength. In order to overcome this difficulty the Geonor penetration vane was used to measure shear strength at 0.75m intervals through the alluvial clays and silts. An example of the results of this test is given in Table I, and generally range between 24 and 58 kN/m² with a sensitivity of approximately 3.5.

In order to give an indication of the density of the non-cohesive soils, standard penetration tests were carried out at 1.5m intervals.

*The Geonor vane apparatus consists of a small cruciform vane attached to a thin steel rod which is pushed into the ground to the depth at which the test is to be made. A torque is then applied gradually to the vane and the maximum torque when failure occurs is recorded and used to calculate the in-situ shear strength of the clay. Immediately afterwards the vane is turned through several revolutions and the test repeated to measure the remoulded shear strength. The ratio of the natural strength to the remoulded strength is known as the sensitivity.

+Standard penetration tests utilise a 50mm external diameter thick-walled sample tube driven into the bottom of the borehole by a 63kg hammer falling freely through 760mm. The tube is driven an initial 150mm to allow for disturbed material. The number of standard blows (N) required to drive the sampler a further 305mm is recorded.
## IN SITU VANE SHEAR STRENGTH RESULTS

### Table I

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<th>Depth</th>
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APPENDIX II

PILE DESIGN

INTRODUCTION

The piles used on the overbridges were 325mm diameter tubular steel piles and those on the underbridges were 305mm x 305 x 110kg steel "H" piles. The Kings Sedgemoor Drain Bridge and the Huntworth Viaduct were the two exceptions, where 600mm and 1.2m diameter bored piles were used.

The design engineers were faced with two problems in the design of the piles, as revealed by the soil survey and experiences emanating from Norway. These were (i) to design a pile to withstand the additional loading imposed by the down drag of soil due to the settlement of the embankments and (ii) to allow in the design for the additional stresses acting on the pile in a lateral direction, again due to settlement and strain.

1. NEGATIVE SKIN FRICTION ON PILES - In the design of the piled foundations for the structures on the M5 Motorway it was apparent that additional loads would be carried by the piles due to the large settlements in the surrounding soft clay.

The soft alluvium extends to 29.0m average depth and overlies the limestones of the Lower Lias, interspersed with weaker layers of clay and mudstone. The down drag force would result in additional settlement, which would be due
partly to the compression of the pile and partly to penetration of the pile point into the bearing stratum.

Work by Johannessen and Bjerrum (1965) has shown that the distribution of the adhesion between clay and pile was governed by the distribution of the effective stresses in the surrounding clay and can be expressed as \( A' \cdot K' \tan \theta' \) where \( A' \) is the effective vertical stress in the soil. The ultimate \( K \tan \theta' \) value used for the soft marine clays was 0.20 and for the lower sands and silts 0.25.

By using a 1mm thick bitumen slip layer on piles Bjerrum, Johannessen and Eide (1969) found that a substantial reduction in the down drag could be achieved. A suitable bitumen for such an application was proposed by Shell International Petroleum Co Ltd to resist the effects of temperature variation and driving stresses. Test piles in the Netherlands where 10mm thick slip layers were used confirmed that the amount of negative skin friction shed was related directly to the thickness and viscosity of the bitumen coating and the rate of settlement of the surrounding soil.

From pile tests carried out on the motorway the ultimate load was established from which the working load/pile was calculated as:

\[
F = \frac{\text{Ultimate load} - \text{Calculated down drag}}{S} \tag{2}
\]
The settlement gauges at the overbridge sites indicated settlements of the order of 1mm/day from which the down drag force was calculated. Figure A shows in tabular form the predicted working loads for piles with various thicknesses of bitumen coating, from which it was decided that a 3mm thick slip layer would be adopted for the 70 ton working load piles.

The design of the bitumen layer is related to the relative viscosity of the bitumen and the rate of settlement of the embankment.

The additional cost of bitumen coating was approximately £20 per pile, 3¾% of the cost of the pile.

ii. LATERAL FORCES ON PILES - Slope indicator readings, see Figure B, confirmed that construction of the overbridge approach embankments caused considerable horizontal movement of the alluvium which would induce high bending stresses in the piles. It was anticipated that a large part of this movement would occur during the first year after building the embankments and so a restriction was included in the contract prohibiting piling for 12 months after placing adjacent embankments, so that ground movements could dissipate. In some cases, the results obtained from the instrumentation readings enabled the 12 month period to be reduced. In order to be sure that the piles would not become overstressed in time it was necessary to estimate the amount of post construction movement likely to occur, and slope indicator records enabled the horizontal movement, at the depth of maximum movement, to be plotted (see Figure C) and extrapolated to indicate the magnitude of future movement to be expected. This diagram confirmed that piles would have been incapable of
Steel piling for overbridges

<table>
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<th>Bridge Number</th>
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</tr>
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FIG. A PREDICTED WORKING LOADS FOR PILES WITH VARIOUS THICKNESSES OF BITUMEN COATING
accepting the full horizontal displacement from the start of construction of the embankments, but it was possible to design piles to deflect with the residual soil movement without over-stressing the pile section.
INTRODUCTION

The design of the bentonite/cement grout mix was examined in order to ensure that, (a) the mix was impermeable enough to seal the piezometer from the overlying strata and (b) was flexible enough to consolidate with the surrounding soil and so avoid the possible penetration of the piezometer if the column of grout were strong enough to act as a pile under load.

MIX DESIGN

The bentonite/cement mixes investigated were:

i. 3 bentonite: 1 cement
ii. 2 " 1 "
iii. 1 " 1 "

After mixing the three samples were placed in glass bottles and were then examined at intervals. One week after mixing the 1:1 mix was in the stiff to hard category and could not be marked by a finger. The 2:1 mix was in the firm to stiff category and the 3:1 mix was the only one to resemble closely the soft alluvial clays.

Thus by visual examination and without resort to laboratory vanes or unconfined compression tests etc. the selection was made.
FIELD TRIAL

It was decided after further field trials to reduce the mixing/placing time to a maximum of 15 minutes. This was decided upon in order to prevent the bentonite from absorbing too much water in mixing and at the same time maintaining the consistency of the mix suitable for pumping.

Following the installation of additional slope indicators with pea gravel used as backfill the writer would prefer to use this in place of bentonite/cement for all slope indicator installations. The tube is held better in the borehole and there is no danger of the grout causing blockages at joints and it also overcomes the problems of corrosion of the casing.
APPENDIX IV

CONSTANT HEAD PERMEABILITY APPARATUS

INTRODUCTION

The unit, as shown in Figure A, is manufactured by Soil Instruments Ltd and consists of a constant head mercury pressure unit which pressurises the de-aired water in one tube of the piezometer through a column of paraffin, the other tube remaining coupled to the manometer recording panel. The paraffin-water interface passes through a 50ml burette in order to measure the amount of water flowing to the piezometer tip.

OPERATION

Soil Instruments Ltd recommend that the two cylinders should be adjusted until the mercury pressure in the lower pot is in excess of the pore-water pressure by 5-10 psi (25 - 50cm of mercury). This involves:

a. Reading the manometer and so determining the pore water pressure which is measured in relation to the back pressure from the header tank.

b. The pore water pressure reading should then be adjusted by the difference in level between the header tank and cylinder B (mercury level).

c. The excess head is then added in and the level of mercury in cylinder A is adjusted accordingly.
THE CONSTANT HEAD PERMEABILITY UNIT - FIG.A
In place of this procedure the writer decided to maintain the manometer reading during the test (in the above procedure the manometer is isolated from the triple valve) and to adjust the cylinder A to produce the required change in pressure on the manometer ie + 3.5 - 7m of water. These pressures were considered to be too high and an excess head of 1.5 - 3m of water was used.

CONCLUSIONS

This method of applying the excess head enabled the expulsion of water from the tip under minimal pressure change and thereby caused little disturbance to the soil structure. At the same time the duration of the test was kept within reasonable limits (average 2 hours/test).

Where the permeability of a soil is high the above equipment has serious limitations if used as designed. Modification to the apparatus as suggested by M. KAY* allows a continuous circulation of water under constant head with very little interruption for valve changeovers. The principle being to use two of the three burettes only and link them to form an inverted "U" tube, pressure being applied to raise the water level in the one tube to provide the pressure and flow from the other. When the 50ml of water has been discharged a quick change over on the valves enables the procedure to be reversed and so on ad infinitum.

*M Kay developed the modification referred to whilst working for the soils, materials consultants, Sandberg Ltd.
APPENDIX V
HORIZONTAL STRAIN CALCULATIONS

1.1 INTRODUCTION
The line of the motorway severed the existing land drainage system, which was reconnected into large drainage channels (rhynes) adjacent to the motorway. In several places these rhynes were interconnected by culverts. The culvert units were generally 3.0 x 2.0 x 1.0m with 105-115mm long joints, see Figure A. The units were laid on a hogging curve to allow for the anticipated settlement.

In two cases during the surcharge period, when the ground loading was approximately double the design loading, failure occurred in the end sections of the culverts resulting in the collapse of a headwall.

2.1 CORRELATION OF DATA
It was apparent that the magnitude of extension exceeded that expected to be induced by vertical displacement caused by settlement of the embankments. Calculations based on work done by Rutledge and Gould (1960) and extended by Cappleman (1967) gave values of horizontal strain per unit of 95mm. The calculations were however developed for triangular or near triangular loadings (ie; dams with narrow top widths 3.5 to 5.5m) on homogeneous or fairly uniformly stratified foundations of finite depth.
Inside CULVERT UNIT (3.0x2.0x1.2m)

Unit weighs approx 2.85 tonnes

Scale 1:20

CULVERT UNIT - FIG. A

JOINT DETAIL

Scale 1:20

Inside of culvert wall

22 mm Ring

105

61

105

96

105

96

GULVERT UNIT - FIG. A
The method of predicting horizontal deformations was based on (i) relating horizontal strain \( \varepsilon_h \) to vertical strain \( \varepsilon_v \) in the foundation, (ii) estimating vertical stresses in the foundation due to surface loading and, (iii) estimating vertical strain in the foundation by reference to the stress-volume change curves \( (e-p) \).

Horizontal strain refers to the average joint extension \( \frac{\varepsilon_h}{L} \) or compression divided by the spacing of the culvert joints \( L \). Vertical strain refers to the maximum observed settlement \( h \) divided by the depth of compressible foundation \( d \).

From these relationships the following equations were developed:

\[
\text{Strain ratio } R_e = \frac{\varepsilon_h}{\varepsilon_v} \quad (1)
\]

In work done by Rutledge and Gould they developed dimensionless influence charts for determining horizontal strain at the base of a triangular embankment for various conditions of elasticity and plasticity. In the evaluation of horizontal strain that part caused by boundary stresses and that part caused by the vertical load are not separated and give rise to the equation:

\[
\frac{C_{\text{cpdh}}}{b^2} \quad (2)
\]

which is referred to as the foundation strain factor where:
Cc - the weighted average compression index
p - maximum unit load of embankment
d - depth of compressible foundation
h - height of embankment
b - base width of embankment

The state of stress in the foundation is indicated by a foundation stress ratio \( \frac{2pd}{s'b} \) where \( s' \) = minimum shear strength of upper stratum.

The authors referred to previously then plotted graphs of strain ratio \( Re \) versus the foundation strain factor \( \frac{Cc pdh}{b^2} \) and correlated the plots with values of the foundation stress ratio \( \frac{2pd}{s'b} \). Where this ratio was equal to or less than one the lines so obtained were referred to as elastic behaviour lines and were represented by the equation:

\[
Re = 0.55 \ e^{-0.1 \left( \frac{Cc \ pdh}{b^2} \right)}
\]

Where they exceeded one, the lines were referred to as the plastic behaviour lines and were represented by the equation:

\[
Re = 0.55 \left( \frac{2pd}{s'b} \right)^{3.73} \ e^{-0.1 \left( \frac{Cc \ pdh}{b^2} \right)}
\]

From this relationship the value of joint extension was calculated as:

\[ \Delta L = Re \times ev \times L = 95 \text{mm} \]
In trying to apply the above formulae to a trapezoidal embankment constructed on a semi infinite mass of heterogeneous soil, see Figure B, certain assumptions had to be made.

i. The height of equivalent triangular loading was maintained at existing embankment height.

ii. No account was taken of the drainage rhynes running parallel to the embankment.

iii. No account was taken of the preconsolidation pressure in assessing \( \bar{C}_c \), the weighted compression index.

iv. The weighted average compression index was determined from Figure C where \( \bar{C}_c \) is plotted against moisture content after Fadum (1941).

v. Equivalent shear strength of the weakest stratum is computed from

\[
S' = 0.5p' \tan \phi' + c' \quad \text{(after Cappleman)}
\]

The last assumption has a considerable influence on the strain ratio \( R_e \) and thereby the calculation of joint extension. Figure D has been plotted to illustrate this variation.

3.1 COMPUTED DATA

In trying to apply the above formulae to a trapezoidal embankment constructed on a semi infinite mass of heterogeneous soil, see Figure B, certain assumptions had to be made.

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EMBANKMENT PROFILE AT THE HUNTSPILL CULVERT — FIG. B
From Figure B

Height ratio = \( \frac{85}{5.035} = 16.87 \)

Depth ratio = \( \frac{85}{17.1} = 4.97 \)

Vertical strain ratio \( ev = \frac{d}{d} = \frac{1.2}{17.1} = 0.0702 \)

Maximum embankment loading = PFA = 2.468 x 14.14 = 34.95

Heavy fill = 2.567 x 21.2 = \( \frac{54.40}{89.35kN/m^2} \)

Equivalent shear strength \( s' = 0.5 \rho \tan \phi' + c' \)

\[ = 0.5 \times 89.35 \tan 32^\circ + 0 \]

\[ = 27.92 \text{kN/m}^2 \]

From Figure C \( Cc = 0.378 \)

\[ . . . \text{Foundation stress ratio } = \frac{2pd}{s^2} = \frac{2 \times 89.34 \times 17.1}{27.92 \times 85} \]

\[ = 1.288 \]

this exceeds the elastic limit of the foundation

Foundation strain factor = \( \frac{Cc pdh}{b^2} \)

\[ = \frac{0.378 \times 89.35 \times 17.1 \times 5.035}{85^2} \]

\[ = 0.403 \]

Strain ratio for "plastic behaviour"

\[ \text{Re} = 0.55 \left( \frac{2pd}{s^2} \right)^{3.73} = 0.55 \times 2.569 \times 0.9608 \]

\[ = 1.358 \]

Horizontal strain \( eh = \text{Re} \times ev \)

\[ = 1.358 \times 0.0702 = 0.0954 \]

Joint extension \( \Delta L = eh \times L \)

\[ = 0.0954 \times 1.0 \]

\[ = 95 \text{ mm} \]
The analysis of embankment loadings on the culverts on the motorway is not an ideal application of Cappleman's work, due largely to the extended base width of equivalent loading and the variable shear strength of the underlying strata.

The results obtained however correlate with observed readings from hilti nails installed either side of the culvert joints on the River Huntspill culvert. The mid-depth average extension in the tension zone is of the order of 100mm, and this is confirmed by readings of strain from the horizontal settlement gauges under similar embankment loadings. The gauge has however proved unreliable for accurate strain reading due to the temperature variation effects on the calibrated cable, which was made from plastic and was subject to extension and compression due to temperature and strain.

Further work is required on this subject to idealise the loading condition represented by a motorway embankment.

Modifications to the horizontal settlement gauge have been suggested which will enable accurate readings of strain to be obtained, namely the inclusion of an invar metallic tape in the tube and a built-in de-airing valve. Figures 18 and 19 show an average settlement and strain profile under a high side road embankment.
APPENDIX VI

SLOPE INDICATOR COMPUTATIONS

INTRODUCTION
At the height of filling operations it was necessary to keep up a constant series of readings on Slope Indicator installations.
In order to save time on calculations which could take up to 1 hour per installation it was decided to prepare a set of tables of horizontal deflections. All dimensions are imperial as the instrument is calibrated in imperial units.

Referring to Table 1 and Figure A
then \( \tan \theta = \frac{(3) - (4)}{2K} \)
where \( \theta \) is the inclination of the casing to the vertical and \( K \) is the instrument constant (2000 in this case)

A change in inclination produces a change in dial difference
\[ \text{ie: } \Delta \tan \theta = \frac{(5) - (2)}{2K} = \frac{1}{2K} \]

The change in lateral movement at an increment of depth is \( \frac{\Delta m}{L} \)
and \( \Delta m = \frac{1}{2} \tan \theta \frac{1}{2K} \)
\[ = \frac{1}{2} \frac{(6)}{2K} \]
where \( L \) is the average distance between readings and \( K \) is the instrument constant.
HORIZONTAL DEFLECTION OF SLOPE INDICATOR TUBING - FIG. A
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Total lateral movement of any point with respect to the bottom
\[ M = \frac{1}{2K} \frac{d}{L} \]  

Consider a 10'0" length of tube. The number of readings (N) per length such that the torpedo does not span a joint equals 9.

\[ l = \frac{L}{N} = \frac{10 \times 12}{9} = 13.3 \text{ (ins)} \]

\[ \frac{1}{2K} = \frac{13.3}{4000} = 0.003 \]

\[ \therefore \text{Horizontal deflection} = (7) \times 0.003 \]

Over the lengths of tubing installed in the alluvium open joints were used to allow for settlement and therefore the lengths of tube to be considered could vary from 10'0" to 10'6" from the top of one tube to the top of another.

Therefore we have:

Horizontal deflection \[ \Delta = \frac{d}{L} M \times \frac{1}{2K} \]

where \( \frac{d}{L} M \) is any whole number

and \[ l = \frac{L}{9} \]

and \( L \) varies between 120" and 126" in increments of \( \frac{1}{4} \) (\( \frac{1}{4} \)" being the limit of accuracy of measurement on the calibrated cable).

\[ \therefore \Delta = \frac{d}{L} M \text{ (variable)} \times \frac{L}{9} \text{ (variable)} \times \frac{1}{4000} \]
This programmed for an Olivetti desk top computer produced a printout headed $\Sigma M$ (values 1 - 100 in increments of 1 and 100-900 in increments of 100) and containing for each value of $L$, a value of $\Delta$

After using this method for some time it became evident that the method of computing deflections, based on individual lengths of tubing, was giving anomalous results at the joints and also did not take into account any silting up of the bottom of the tube. The latter can affect up to 5'0" of tubing. In order to overcome this a further set of computations were made using $L$ as the total length of the installation and $N$ as the total number of readings obtained throughout the installation.

Individual differences between this method and the previous one were found to be small and did not necessitate alterations to previous plots. A much smoother plot was however obtained.

A printout can again be obtained using the Olivetti but we now have

$$\Delta = \Sigma M \text{ (variable)} \times \frac{L \text{ (variable)}}{N \text{ (variable)}} \times \frac{1}{4000}$$

and in this case the printout contains a value of $\Delta$ for each value of $L$ but is headed by $N$ in addition to $\Sigma M$. 
APPENDIX VII

PERFORMANCE OF THE INSTRUMENTATION USED
TO MONITOR THE M5 MOTORWAY EMBANKMENTS BUILT ON
SOFT GROUND BETWEEN EDITHMEAD AND HUNTWORTH

John M McKenna¹ and Malcolm Roy²

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INTRODUCTION
For 35km, the M5 motorway crosses the Somerset Levels where the
depth of soft alluvium is typically about 26m. Because of the poor
ground conditions, the stability of all the embankments was low,
and large settlements were expected. To reduce the post construc­
tion settlement of the motorway, the embankments were surcharged
and allowed to settle for one year. The side road embankments which
could not be surcharged because of stability considerations, were
built to the design height as rapidly as the strength of the ground
permitted, and allowed to settle for up to a year before construction
of the bridges started. This paper discusses the performance of the
instrumentation used on the southern half of the 35km length. A
general paper on the monitoring of the whole 35km appears elsewhere
in the proceedings.
EMBANKMENT INSTRUMENTATION

The object of the instrumentation was

1) to enable the actual vertical and horizontal movements of the foundations of the embankments to be measured in order to check the stability and the settlement and surcharge design calculations;

2) to measure the excess pore water pressure build up during construction, and the subsequent dissipation during the settlement period. From these records the factor of safety of the embankments was calculated in terms of effective stress, and the settlement calculations were checked.

Rod settlement gauges, inductive settlement gauges, piezometers, horizontal strain and settlement gauges, slope indicators, and toe pegs were used. See Fig 1. The number of instruments used is given below:

- Rod settlement gauges: 362
- Perforated settlement gauges: 76
- Inductive settlement gauges: 4
- Piezometer huts: 28
- Piezometers: 342
- Horizontal strain and settlement gauges: 6
- Slope indicators: 23
- Toe pegs: 1270

The costs referred to below are based on the tender submitted in January 1971.
General Arrangement of Instrumentation.

Fig 1
Details of the two types of gauge are shown in Fig 2. The rod settlement gauges were installed at 100m intervals on the motorway centre-line and at 200m intervals on the hard shoulder, offset from the centre-line by 17.3m. They were staggered in order to provide two gauges at every 100m along the motorway. They were originally designed with a concrete base to be cast in-situ, but this was later changed to a 600mm square steel plate, 12mm thick, with the first section of the rod welded into a hole in the centre of the plate, so that the lower half was a spike which was driven into the topsoil to fix the gauge. The rods were sleeved off from the surrounding fill by 50mm diameter pitch fibre pipes in order to prevent any down-drag force due to compaction. To protect the gauge during filling, it was specified that a 750mm diameter dry stone wall should be built around the gauge. During construction, however, this was found to be unnecessary, and it was later omitted.

Two perforated settlement gauges were positioned over each piezometer group, allowing the standing water level in the fill to be measured. The outside of the 40mm diameter pipe was coated with bitumen to reduce the down-drag of the fill due to compaction.

All the gauges were extended by the main Contractor as required. The overall cost of each type of settlement gauge was about £35. The cost of the water dipper was £26.
The main objections to the rod settlement gauges are that instruments projecting above the fill interfere with the placing of the fill and are liable to be damaged by the plant. It is difficult to be specific about how much the RSG's interfered with the construction of the embankments. The major effects were probably the additional compaction required around each gauge, and the difficulties the graders had in getting the windrows past them when maintaining the formation. To provide against damage by plant, a diamond shaped board 600 x 600mm was placed over each gauge as a warning. In spite of this, about 25% were damaged, and bent rods had to be replaced.

**INDUCTIVE SETTLEMENT GAUGES**

These were used to measure the settlement at different depths below original ground level. The gauge consists of 25mm and 38mm rigid PVC telescopic tubing installed in a 200mm diameter borehole to bedrock, details of which are given in Fig 3.

At 3m intervals, expanding plates were keyed into the sides of the borehole so that they were independent of the telescopic tubes. The hole was then backfilled with 3:1 bentonite:cement grout. Using an induction coil in a probe, the depth of each plate below a datum point, and hence the settlement, could be determined.

There is a basic difficulty in installing this type of instrument in sensitive soils, in that the expanding plates are supported by
the soil on the perimeter of the borehole, and if this becomes disturbed when the borehole is sunk, it may not have sufficient strength to support the plate. In such a case, unless the grout is stiff enough to support it, the plate falls down the borehole until it takes up the slack in the tubing which has been used to pressurise the expanding mechanism.

At first, some trouble was experienced with the probe which was not watertight. The lead was found to need careful handling, as a kink could break the electric cable.

Those gauges, being made of pvc, were easily damaged by earth moving plant, and, once damaged, were difficult to reinstate, as fill material could then fall down the tubing and lodge on one of the telescopic sections. The four gauges installed on this length of motorway were damaged beyond repair in this way early on in the Contract.

The average cost of an inductive settlement gauge 24m long was about £380, and the cost of the probe was £112.

**PIEZOMETERS**

Groups of piezometers were placed under the motorway at about 600m intervals and on both banks of the three river crossings and under one approach embankment of each side road crossing. The first piezometer in each group was installed in a 1m long sand pocket at
the bottom of a borehole from which continuous 54mm diameter piston samples had been taken. These samples were extruded in the site laboratory, cut into 150mm lengths and split open lengthways, one half being used for moisture content and Atterberg Limit tests, the other being laid out on corrugated polythene trays to air dry. When partially dry, the samples were described in detail, in order to fix the depths of the piezometers for the rest of the group. The piezometers used had a low air entry value and a permeability of about $3 \times 10^{-4}$ m/s. Details are given in Wilkes (1970). They were installed in 75mm diameter flight auger holes which were bored to 300mm above the required depth, then pushed into position on the ends of detachable rods, and the holes were backfilled with a 3:1 bentonite:cement grout which was tremied in.

**Installation and De-airing**

Each piezometer was connected to a 12m length of twin tubing and installed dry as a construction expedient. The 12m length of lead provided a check on the depth of the piezometer below ground level. The leads were subsequently joined to leads from the instrument hut which had been filled with de-aired water. With one of the joints undone, water was forced under a low pressure down the other lead into the tip and up the return until it started to flow from the undone joint. With water weeping from each, the leads were joined and the complete circuit was de-aired using a fairly high positive pressure on one lead and a small vacuum on the other. Within a day or two, however, the mercury manometers gave out of balance readings,
and it was realised that the initial de-airing pressures had generated a pressure at the tip in excess of the hydrostatic pressure, and that, as a result, some of the original air and water in the leads had been forced into the soil surrounding the tip. This problem arose because the Contractor, when using the standard formulae to calculate the de-airing pressures, did not take into account that the piezometer tip was below the instrument hut rather than above it, which is more usual. The de-airing pressures were then changed so that the pressure inside the tip equalled the hydrostatic pressure in the soil. De-airing using these pressures took longer.

As the settlements under the embankments were going to be in the order of 1 to 2m, it was desirable to use a grout backfill for the piezometer holes which had similar compressibility and shear strength characteristics to the natural ground. If a strong grout were used, the column of backfill would act as a pile and force the piezometer down with the settlement of the embankment. Simple tests carried out on site indicated that the most suitable grout was probably the specified 3:1 bentonite:cement mix, with the mixing time kept to a minimum (15 minutes). A research project into the shear strength and compressibility characteristics of grout backfill would be to the benefit of the profession.

The leads from the tips were colour coded at 1m intervals from a point 2m within the embankment to the instrument hut. It would have
been better to have colour coded the leads for the complete length, as several were cut along the centre-line of the motorway during drainage excavations. Without positive identification, pairing up the severed leads caused considerable problems. Only three piezometers, that is 1%, became unsatisfactory during the Contract.

Constant head in situ permeability tests were carried out on all piezometers prior to embankment loading, on completion of loading, and prior to surcharge removal, the applied heads being kept low. (Gibson 1963, 1966, 1970, Wilkinson 1968 and Bjerrum et al 1972). Because of the differences in level between the manometer unit, the header tank and the permeability test apparatus, the applied head was measured using the manometer on the piezometer board itself. The cost of making and backfilling a hole for a piezometer was approximately halved by the use of a flight auger instead of a shell and auger. The average cost of a piezometer in a flight augered hole was £125 and in a shell and auger hole £165. The additional cost of piston sampling, including the cost of the equipped site laboratory and store, was about £150 per hole.

**HORIZONTAL STRAIN AND SETTLEMENT GAUGES**

Six gauges were installed to measure the strain and settlement profile under selected embankments. Each gauge has a 48mm outside diameter plastic tube laid in a trench 600mm below original ground level. Metal plates, approximately 500 x 100mm, were threaded on
to the tube at 3.05m centres. The trench was backfilled with sand or a fine granular fill and the ends located in concrete blocks with lockable covers. See Fig 4.

The settlement recording equipment consisted of a liquid line connected to a transducer housed in the cable drum, which measured the pressure changes directly in terms of settlement. The settlements were always read at the same points in the tube which were fixed relative to the metal plates along the tube. The plates were located using a probe containing a coil which formed the active arm of a parallel resonance bridge.

As the gauge was still being developed, several teething problems arose. The main ones on the settlement side were that the liquid line needed frequent de-airing and that it was subject to temperature changes. The horizontal strain measurements were of little value as the inaccuracies in the distance measurements were greater than the movements of the plates due to the embankment loading. The gauge has since been modified to facilitate easier de-airing, and a steel band is now used to measure the plate distances.

**SLOPE INDICATORS**

Slope indicators were installed at the three river crossings, on the northern approach to the Huntworth Viaduct, on one of the side road embankments and at four bridge sites to measure the lateral movements below ground. The extruded 80mm diameter aluminium tubing
was installed in a 200mm diameter borehole sunk 0.3m into bedrock. The tubing was supplied in 3.048m lengths which were joined by external couplings. Gaps varying from 25 to 150mm were left between adjacent lengths of tube to allow for settlement. The annulus was backfilled with 3:1 bentonite:cement grout and the tube was capped with a steel pipe fitted with a lockable cover, the pipe being anchored in a concrete block.

The recording unit was powered by four 6 volt batteries. The aluminium body of the torpedo became scored during use, as it rubbed against the inside of the tubing, and slightly pitted at the junction with the stainless steel cap, due, it is thought, to electrolytic action. There was some evidence of the aluminium tubing being attacked in the ground, frothing being noticed in three of the tubes.

As the cable between the recording unit and the torpedo requires careful handling, a cable drum was specified in the Contract.

The instrument has been used continuously for over two years on site, and during this time has only required one set of replacement batteries and the repair of two minor electrical connections. For these repairs, however, the instrument had to be sent off site. At the beginning, each tube was read twice and these readings confirmed that the instrument was accurate to 6mm in 30m. It was found that one pair of grooves 25m long took one hour to read in the field and another hour in the office to reduce the readings.
and plot the deflections. To facilitate the reduction of the readings, a table converting readings to deflections was prepared on site.

The cost of the slope indicator torpedo, cable, cable drum and recorder was about £2,000, and the cost of a tube installed in a 24m deep borehole about £360.

TOE PEGS
These were installed at 20m intervals along the toes of all embankments where potential failure conditions were thought to exist. This was for all side road embankments where the height of heavy fill exceeded 4m, and for the surcharged motorway embankments where the 2m deep drainage ditch was close to the toe of the embankment.

The pegs were controlled by survey lines 30m offset from the embankments. At each reading, the 30m offset was checked, and a theodolite used to determine the lateral movements of the pegs. Levels were also taken.

The pegs were 50 x 50 x 750mm timber stakes driven into the ground and surrounded by concrete. Errors arose because it was often two weeks after the stakes were driven before they were concreted in. To do this, the ground was cut away from around the top of the peg, and this caused some disturbance. Some of the stakes became loose in summer as the surface crust dried out, showing that where readings
are required for several months during the summer, stakes longer
than 750mm are advisable. Each toe peg cost £2.50.

**SETTLEMENT PEGS**

For measurement purposes, settlement pegs were placed on top of
all the embankments at the start of the settlement period. These
were spaced at 40m intervals along the centre-line and hard shoulder
of the surcharged motorway and at 20m intervals along the verges of
the side road embankments. They were levelled twice, at the beginning
and at the end of the settlement period.

**STAFF**

When embanking was in progress, seven men were employed on monitoring.
These were a section engineer, an assistant engineer, a graduate
engineer, a draughtsman, and three technical assistants. With
several side road embankments under construction at the same time
as the motorway embankment itself, continuous effort was required
by these men to keep pace with the Contractor.

**CONCLUSIONS**

The instrumentation in general performed very well and the motorway
and side roads were built without any failures. The construction
of several side road embankments was stopped because of high pore
water pressures in the foundations, and only started again when
the effective stress analysis indicated that it would be safe. One
embankment, where the pore pressures were not dissipating fast
enough, required the addition of wide berms to allow it to be constructed to full height.

The only instruments which did not perform satisfactorily were the inductive settlement gauges, which became blocked with fill after being damaged by earth moving plant, and the horizontal settlement and strain gauges. These were in the course of development, and although the settlement gauge worked well, the horizontal strain measurements were not sufficiently accurate to allow the ground movements to be measured.

ACKNOWLEDGEMENTS

The paper is presented with the permission of Mr P G Lyth, the Director of the South Western Road Construction Unit, Engineer to the Contracts, and Mr A S Turner, Chief Engineer of the Somerset Sub-Unit of the SWRCU. The authors wish to thank them.

They also wish to thank, amongst many others, Mr B A Heskins and Mr A R Croall, for their assistance in taking the many readings.

REFERENCES


McKenna, J M (1973) Monitoring the M5 motorway embankments on soft ground in Somerset. BGS Symposium on Field Instrumentation.


APPENDIX VIII

STABILITY ANALYSES

INTRODUCTION

Typical stability calculations for a side road approach embankment are detailed. The three methods of analysis used namely

- a) Total stress analysis
- b) Effective stress analysis
- c) General Stability analysis, are considered, together with the computer analyses used to check the results derived from method (b).

(a) TOTAL STRESS ANALYSIS

The selection of the appropriate value of undrained shear strength (Su) for the Somerset clay required experience and judgment, as the unconfined compression test results are significantly lower than the Geonor vane results.

The shear strength for the saturated clays under undrained (φ = 0) conditions can be represented by:

\[ Su = \frac{1}{2} qu = c \]

where \( qu = \) the compressive strength of the clays
\( c = \) cohesion

The average minimum values of undrained shear strength (Su) from the unconfined compression tests and in situ vane tests were 16.8 kN/m\(^2\) and 31.2 kN/m\(^2\), respectively,
The back analyses of a number of case records of embankment failures on the Bristol Channel alluvium indicated a value of \( \mu_u \) of 21.6 \( \text{kN/m}^2 \). This value has been used in the calculations.

The probable position of the surface of sliding has been determined from the investigation of some of the case records referred to above. The general indication being that the surface of sliding is located between 4.9m and 7.3m below ground level.

The probable surface of sliding is represented in Fig A by a circular arc of radius \( R \) and its centre at point 0.

The calculation of the disturbing and restoring moments are set out in the following table.

<table>
<thead>
<tr>
<th>Slice No</th>
<th>Area of slice ( A ) (m(^2))</th>
<th>Unit weight of soil ( \gamma ) (kN/m(^3))</th>
<th>Weight of slice ( A \gamma ) (kN/m)</th>
<th>Lever arm ( x ) (m)</th>
<th>Disturbing Moment ( A \gamma x ) (kN.m)</th>
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WITHY ROAD OVERBRIDGE EMBANKMENT

TOTAL STRESS ANALYSIS OF A SIDE ROAD EMBANKMENT - FIG.A
<table>
<thead>
<tr>
<th>Slice No</th>
<th>Area A (m²)</th>
<th>Unit weight of soil (kN/m³)</th>
<th>Weight of Slice A (kN/m)</th>
<th>Lever arm (m)</th>
<th>Disturbing Moment A (kN.m)</th>
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</tr>
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<td>1.2 x 1.2/2</td>
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<td>11.86</td>
<td>-12.4</td>
<td>-835</td>
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<tr>
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<td>67.54</td>
<td>67.54</td>
<td>11408</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
(i) Strength of embankment fill included (ie no tension crack considered).

Then restoring moment:

<table>
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<tr>
<th>Soil type</th>
<th>Length of arc</th>
<th>Su</th>
<th>SuRe</th>
</tr>
</thead>
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<tr>
<td>Embankment fill</td>
<td>16.9 x 0.4712</td>
<td>26.4</td>
<td>210.23</td>
</tr>
<tr>
<td>Alluvial clays and peat</td>
<td>16.9 x \frac{110.5}{360} x 2</td>
<td>21.6</td>
<td>704.01</td>
</tr>
</tbody>
</table>

\[ 2 \text{SuRe} = 914.24 \]

Factor of safety for slip surface shown = \[ F = \frac{2 \text{SuRe}}{2 \text{A}x} \]

\[ \therefore F = \frac{914 \times 16.9}{1140.8} = 1.35 \]

(ii) Strength of embankment fill neglected (ie tension crack assumed).

Then restoring moment becomes \[ 2 \text{SuRe} = 704.01 \times 16.9 \]

and \[ F = 1.04 \]

*The embankments were constructed predominantly in PFA and lias clay fill, for which there were no undrained shear strength results available. The following parameters were recommended by J. M. McKenna for use in stability analyses and these have been related to a total stress condition:

<table>
<thead>
<tr>
<th>Soil</th>
<th>Density</th>
<th>Cohesion</th>
<th>( \tan \phi' )</th>
<th>( \phi' )</th>
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</thead>
<tbody>
<tr>
<td>PFA</td>
<td>14.14</td>
<td>0</td>
<td>0.510</td>
<td>27 degrees</td>
</tr>
<tr>
<td>Lias Clay</td>
<td>21.2</td>
<td>0</td>
<td>0.466</td>
<td>25 degrees</td>
</tr>
</tbody>
</table>

\[ \therefore S \text{ for PFA} = p \tan \phi' = \frac{7.2 \times 14.14 \times 0.510}{2} \]

\[ = 26.4 \text{ kN/m}^2 \]
(b) **EFFECTIVE STRESS ANALYSIS**

The stability analyses referred to in Table IV and those referred to later in this Appendix, namely the Genesys computer programme, and the Hewlett Packard programme, have considered a circular surface of failure analysed using the Bishop Simplified analysis, (1955).

The factor of safety is defined as the factor by which the shear strength parameters in terms of effective stress $C'$ and $\phi'$, can be reduced before the slope is brought into a condition of limiting equilibrium, and can be expressed as:

$$ F = \frac{\frac{c'}{m} + \frac{(p - u) \tan \phi'}{m \alpha}}{\frac{p \sin \alpha}{m \alpha}} $$

where $p$ is the head of soil and $u$ the head of water subtended by each strip arc, and $\alpha$ is the angle between the vertical and the total stress normal to each strip arc.

The expression is for equal strip widths and neglects the effect of the vertical shear forces between the strips, which may be equated to zero with little loss in accuracy.

In the equation the factor $m \alpha = \frac{1 + \tan \alpha \tan \phi'}{F} \cos \alpha$

and introduces two values of $F$ which requires a process of iteration before the equation can be solved.

In order to aid this process a family of curves relating $m \alpha$ to $\alpha$ and to $\frac{\tan \phi'}{F}$ have been prepared by Bishop and enables a value of $F$ to be assumed from which values of $m \alpha$ for each
strip are obtained. The value of $F$ calculated is then used in the same way until convergence occurs. This is usually fairly rapid and may involve no more than one iteration.

The section of the Withy Road embankment shown in Fig B has been amended from Fig A to show equal width slices and to indicate the measured excess pore water pressures under the embankment, immediately prior to stopping filling.

The stability calculations for the section shown in Fig B are tabulated below:

<table>
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<tr>
<th>Strip No</th>
<th>$\alpha$ (deg)</th>
<th>$\sin \alpha$</th>
<th>$P$ (kN/m$^2$)</th>
<th>$u$ (kN/m$^2$)</th>
<th>$p$ sin $\alpha$</th>
<th>$c' + (p-u)$ tan $\phi'$</th>
<th>$m \alpha$</th>
<th>$c' + (p-u)$ tan $\phi'$</th>
<th>$m \alpha$</th>
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<td>0.3x21.2 = 6.36</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>6.6 14.14=93.32</td>
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<td>5</td>
<td>-22</td>
<td>-0.375</td>
<td>4.8x17.3 =83.04</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.4x11.8 =16.52</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>99.56</td>
<td>81.4</td>
<td>-37.30</td>
<td>9.66</td>
<td>0.77</td>
<td>12.55</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>-42</td>
<td>0.669</td>
<td>3 x 17.3 =51.9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>30.4</td>
<td>-34.73</td>
<td>11.43</td>
<td>0.46</td>
<td>24.85</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

$\pm 146.56 \pm 184.12 \pm 202.77$
WITHY ROAD OVERBRIDGE EMBANKMENT

Provisional Surcharge

Embankment at day 49

Free Draining Layer

Piezometric Level

EFFECTIVE STRESS ANALYSIS OF A SEDGE ROAD EMBANKMENT – FIG. B
For first approximation \( F_{\text{mol}} = \frac{c' + (p-u) \tan \phi'}{\frac{2}{2} p \sin \alpha} \)

\[ = \frac{184.12}{146.56} = 1.26 \]

then \( F = \frac{c' + (p-u) \tan \phi'}{m \alpha} \)

\[ = \frac{2}{2} p \sin \alpha \]

\[ F = 1.38 \]

The solution of this problem considers the part played by the strength of the embankment fill.

Assuming a tension crack to have developed then,

\[ F = 1.03 \]

**COMPUTER ANALYSES**

In order to improve on the time spent in calculation and to permit a larger number of failure surfaces to be considered, a Hewlett Packard 9800B desk top computer was programmed to analyse embankment stability using the Bishop Simplified method.

The programme was compiled from work done by Pimenta (1970) using the Bishop Simplified analysis, where the factor of safety \( F \) is expressed as

\[ F = \int \frac{k_5}{1 + k_3/F} \frac{k_2 \sec \theta}{} \]
\[ k_2 = W \sin \theta \]
\[ k_3 = \tan \theta \tan \phi' \]
\[ k_4 = (w-\mu b) \sec \theta \tan \phi' \]
\[ k_5 = (k_1 + k_4) \sec \theta \]
\[ f_0 = \text{correction factor which varies between 1.0 and 1.3} \]

The constants \( k_1 \) to \( k_5 \) have been used to minimise the storage requirements in the computer. The programme instructions were kept to under two hundred and the number of stores required does not exceed fourteen.

The analyses referred to so far involve a large collection of data in order to examine an individual surface of failure. Because of this time consuming and rather tedious process, it is usual to find that a limited number of failure surfaces are examined.

In order to overcome this danger the results of the effective stress analyses have been checked against the results obtained from the GENESIS* Slip Circle/1 programme (1974). This programme is run through a terminal connection in Taunton, linking with the main computer in London. The input information is compiled on punched cards and is capable of being edited to suit any change in data.

*GENESIS is the name given to a computer centre based at the University of Technology, Loughborough and stands for General Engineering Systems. The slip circle programme, referred to as Slip-Circle/1, was written by Geocomp UK Limited and is based on the Bishop method of analysis.
At the time the embankments were being constructed on the motorway, it would have been possible to install a terminal in the site offices for approximately £2,000. The results from the instrumentation could then have been interpreted directly into a computer program, with the ability to examine a large number of failure surfaces quickly and print out the results.

The editing facility available with the Genesys programmes would also have overcome the necessity to repunch the data input cards for successive changes in data.

With the experience available from the construction of the motorway embankments, it is clear that some form of computer terminal or desk top calculator would prove invaluable for the interpretation of data and the control of earthworks.

In the Slip Circle/1 programme the actual equation used is:

\[
F = \frac{1}{\sum w \sin \alpha} \left[ \left( c' b + \tan \phi' (W(1 - B) + X_n - X_{n+1}) \right) \frac{1 + \tan \phi' \tan \alpha}{\sec \alpha} \right]
\]

where \((X_n - X_{n+1})\), the vertical shear forces, are assumed equal to zero.
The failure circles to be examined can be specified by presenting a data table containing grids of circle centres and using a command to select a point through which all the circles are to pass or a line which they are to touch. Alternatively a horizontal line of centres can be specified with a set of radii to be tried at each centre.

The results of the analyses using excess pore water pressures measured under the centre line, hardshoulder and toe of the embankments, give factors of safety in excess of those calculated by any other method and are considered by the writer as a more realistic assessment of the stability of the embankments.

(c) GENERAL STABILITY ANALYSIS

The general form of analysis adopted for use on site requires a knowledge of the excess pore water pressures and the loading from the embankment.

The calculations for the section of the Withy Road embankment shown in Fig C are set out below.

The distribution of pore water pressure at Day 49 is overplotted on Fig 34. The percentage of the centre line pore pressure is less at the half width of embankment than for curve A, which was used in the preparation of the Charts in Fig 33. Consequently an adjustment is necessary to the actual centre line pore pressures to bring them in line with the distribution curve A.
The actual distribution at the half width is 73% of the centre line, whereas it should be 85%. An adjustment of the centre pore pressures of \( \frac{73}{85} = 86\% \) is therefore necessary. This produces the plot shown in Fig C marked, "assumed".

The relationship between the degree of consolidation at a given depth and the time factor \( (Tv) \) can be expressed as

\[
U = (\%) = f(Tv \frac{Z}{H}).
\]

Taylor (1948) expressed this relationship graphically and from this McKenna (1971) produced the plot of excess pore pressures against average degree of consolidation shown in Fig 35. The average degree of consolidation on the centre line for the assumed pore pressure with depth plot is

<table>
<thead>
<tr>
<th>Applied load</th>
<th>Heavy fill: 0.4 x 21.2 = 8.48</th>
<th>PFA: 6.6 x 14.14 = 93.40</th>
</tr>
</thead>
<tbody>
<tr>
<td>Free draining layer: 0.3 x 21.2 = 6.36</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\[
108.24 \text{ kN/m}^2.
\]

In terms of equivalent pore pressure, \( \frac{108.24}{9.807} = 11.03 \text{m} \)

Area of the excess pore pressure plot to 7.3m

\[
= 1 \times (1.7 + 3.0 + 3.9 + 4.8 + 5.4 + 5.9 + 6.2) + (0.3 \times 6.4)
\]

\[
= 32.82
\]

\[
\therefore \text{Average degree of consolidation } U (\%) = \frac{(11.03 \times 7.3) - 32.82}{(11.03 \times 7.3)}
\]

\[
= 59\%
\]
From Fig 35 the % pore pressure $U_s$ at $\frac{1}{3}, \frac{2}{3}, \frac{4}{3}$ of 7.3m can be established as:

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>%</th>
<th>$p_{wp}$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.912</td>
<td>13% $U_s$</td>
<td>1.44 $U_s$ is assumed</td>
</tr>
<tr>
<td>2.738</td>
<td>37%</td>
<td>4.08 equal to 5.85</td>
</tr>
<tr>
<td>4.570</td>
<td>53%</td>
<td>6.84 the applied</td>
</tr>
<tr>
<td>6.380</td>
<td>62%</td>
<td>7.07 load</td>
</tr>
<tr>
<td>7.300</td>
<td>64%</td>
<td></td>
</tr>
</tbody>
</table>

The deduced curves from the above distribution are also shown on Fig C, for the centre line, hard shoulder and toe piezometer profiles.

The deduced profiles can be seen to compare well with the actual profiles and give an average degree of consolidation on the centre line of:

$$F = \frac{(11.03 \times 7.3) - 33.42}{(11.03 \times 7.3)} = 59\%$$

... from Fig 33 $F = 1.24$.

The process of adjusting the pore pressure distribution curve to relate to the actual pore pressures under the hard shoulder has to be done with some caution. The pore pressures at the toe of the embankment are critical in this assessment and had the toe pressures at Withy Road been higher, then Curve B in Fig 33 should have been used. With the other piezometer profiles remaining the same, this would have given a factor of safety $F = 1.09$. The more realistic value is that derived from Curve A distribution of excess pore water pressure.
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