Third A-NZ Young Geotechnical Professionals Conference, Cameron, Collingwood & Slatter (eds), Feb. 1998, Melbourne, Australia.

Case History of an Unusual Embankment Failure

Andrew Campbell, Maunsell Pty Ltd

Summary: The partial failure of a 7.5m high surcharged embankment constructed over soft clay and peat is described. Extensive instrumentation within the embankment and its foundation gave only partial warning of the impending failure. Failure occurred during the final lift of surcharge and resulted in a 0.6m wide tension crack approximately 100m long down the centre line of the embankment. Unusual features included the fill material, lack of pore pressure dissipation during construction despite installation of wick drains, limited lateral deflection prior to failure, near perfect symmetry of the failure and the remedial measures implemented.

1 INTRODUCTION

This paper describes some of the problems encountered and remedial measures implemented during construction of a 7.5m high surcharged section of embankment forming part of the 18km long A16 Spalding to Sutterton Improvement Scheme in the Fenlands of eastern England, 160km north of London, refer Figure 1. Embankments up to 6m high were necessary to cross the River Welland north and south of Spalding. Construction of the southern crossing took place over approximately 10m of soft to very soft silty clay and a thin layer of peat. The ground conditions necessitated the use of wick drains and surcharging to accelerate pore pressure dissipation and settlement. Instrumentation was installed within the embankment and its foundation to monitor stability during construction; it comprised hydraulic piezometers, inclinometers, magnetic extensometers, hydrostatic profile gauges and settlement plates. The failure occurred during the final lift of surcharge at the southern crossing, however the full extent of cracking did not become apparent until removal of the surcharge 12 months later. Remedial measures included backfilling the crack with foam concrete and substantially increasing the amount of reinforcement in the concrete roadbase along that section of embankment.

2 PROJECT DESCRIPTION

2.1 Scope of Works

The A16 Spalding to Sutterton Improvement Scheme involved construction of a single carriageway highway across 10km of green field site to the east of Spalding, followed by 8km north along a disused railway embankment to Sutterton. Embankments were generally constructed less than 1.5m high for reasons of aesthetics, settlements, land-take and construction volumes. However, on the approaches to the River Welland and across the flood detention basin of Cowbit Wash, it was necessary to construct embankments up to six metres high. The project chainage ran from Ch 400 in the south to Ch 18,500 at the north end. The area of interest lies between Ch 1100 and 1250.

2.2 Site Description

Spalding is located in the Fens, a low lying region of peat bogs, estuarine mud flats and salt marshes located in eastern England. The Fens cover an area of approximately 4000 square kilometres and have been reclaimed by construction of seawalls and drainage channels since Roman times, however

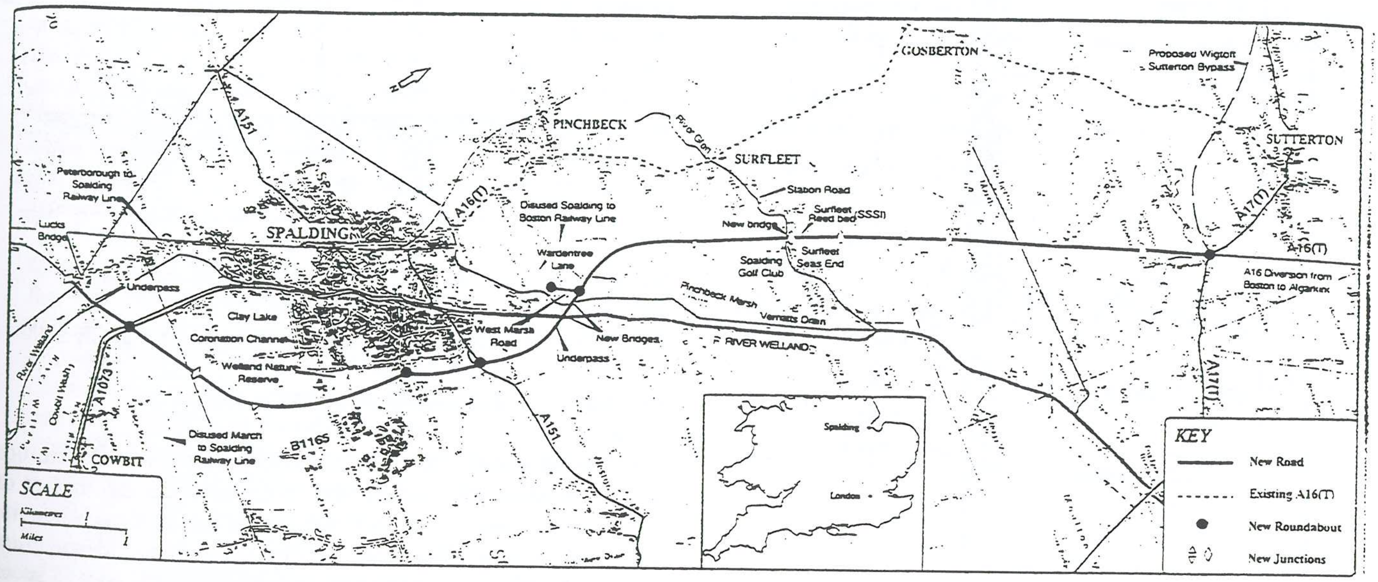

Figure 1: Site Location Plan

most activity has occurred over the past 300 years. The region is typically between 0 – 5m above sea level.

2.3 Geology

The solid geology underlying the Scheme comprises Oxford Clay, a very stiff to hard silty clay of middle Jurassic age. During the Quaternary, the area was subjected to as many as four episodes of glaciation, the last occurring in the Devensian Stage between 70,000 and 10,000 years ago. This resulted in deposition of the Chalky Boulder Clay, a lodgement till comprising a stiff silty clay matrix with gravel sized clasts of chalk and limestone. The Fen deposits overlie the Boulder Clay and comprise peat, clay, silty sand and sandy gravels according to the environment of deposition. The peat beds under the Scheme were up to 1m thick and formed in a freshwater or brackish environment. The silty sands were deposited under marine conditions whilst the clays are inferred to be of estuarine origin. Fluvioglacial sands and gravels are present under parts of the Scheme and were deposited by melt waters either under the ice sheets or as the ice retreated.

The ground conditions immediately under the failed section of embankment are shown on Figure 2 and comprised a 1m thick desiccated crust of firm silty clay, 5m of soft to very soft silty clay with traces of peat, 1m of soft fibrous peat, 1m of soft to firm silty clay, 6m of stiff Chalky Boulder Clay then Oxford Clay.

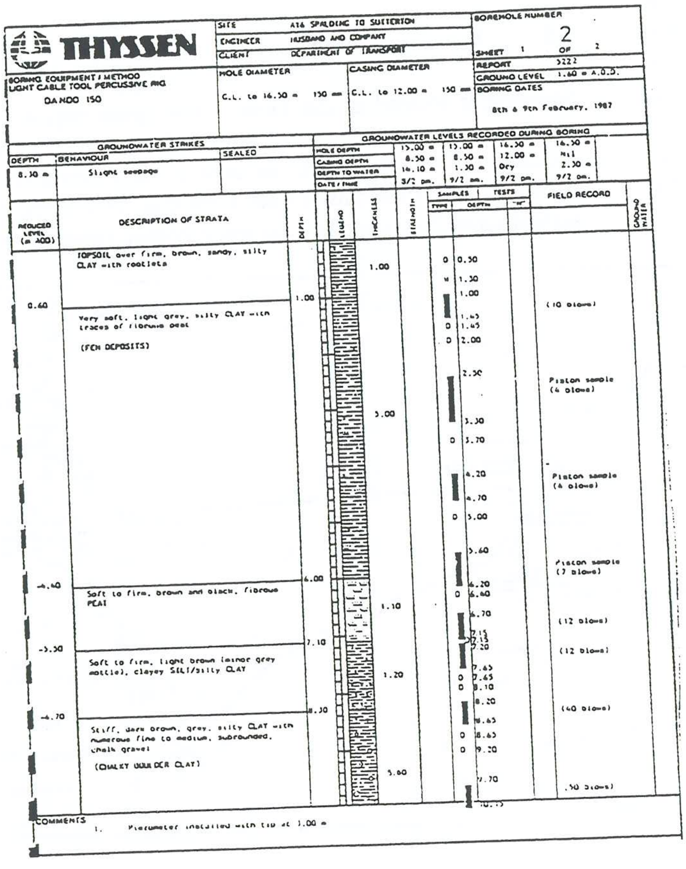

Figure 2: Borehole Log at Ch 1200

2.4 Embankment Design

An embankment height of 6m was required to give clearan across the Welland River. Due to the compressible nature the Fen deposits, a surcharge of 40kPa was required in t form of 2.0m of compacted Class 6F1 sand and gravel. T embankment footprint was 48m wide comprising a formati width of 11.3m and batters set at 1:3. Following topsoil stri a drainage blanket-cum-work platform 0.5m thick was plac over a non-woven geotextile and the instruments installe Wick drains were installed at 1.5m centres through t blanket to the base of the Fen deposits, except in t immediate vicinity of the instruments so as not to cau damage. Uniformly graded granular material was specifi for the body of the embankment. The rate of filling w restricted to 0.75m per seven consecutive days.

2.5 Instrumentation

Instrumentation at Ch1200 comprised 24 hydrau piezometers, five magnetic extensometer settlement gaug four inclinometers, one hydrostatic profile gauge and settlement plate. The arrangement of instruments is shown Figure 3. All piezometers were connected to terminals on t right hand side of the embankment.

The piezometers, inclinometers and hydrostatic profile gau were read using data loggers which down-loaded directly to PC and onto spreadsheets. The magnetic settlement gaug were read using a manual reed switch probe whilst t were read using a manual reed switch probe whilst t settlement plates were surveyed; this data was entered or spreadsheets manually.

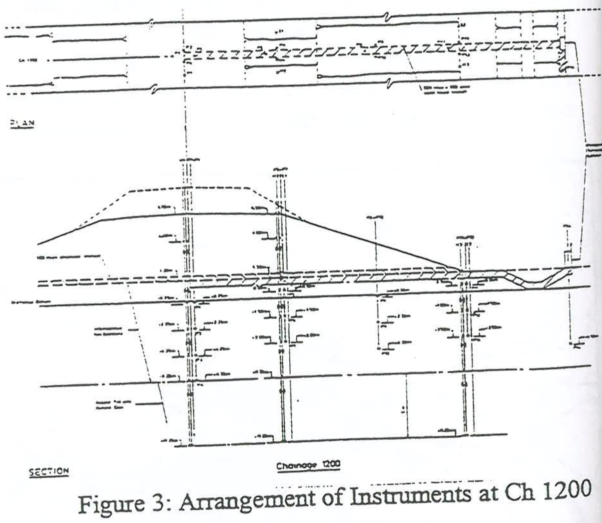

Figure 3: Arrangement of Instruments at Ch 1200

2.6 Embankment Fill Materials

Embankment fill material could not be sourced from bor pits adjacent to the site due to the soft saturated nature of near-surface soils and restrictions on land-take due to area's agricultural use. To enable year-round constructi and hence minimise construction time, material was specif as select granular fill. The Contractor elected to use re bricks from brickworks located 25km south of the site. T material had a bulk density of 1750 kg/m^3 when placed and compacted. The surcharge had a compacted density of 2000 kg/m^3

3 CONSTRUCTION AND MONITORING

3.1 Sequence of Events

Installation and commissioning of instruments was completed at the end of March 1993. The instruments were monitored on a regular basis throughout the construction and surcharge periods. Records for the centreline, right hand verge, shoulder and toe, and fence line piezometers are attached as Figures 4 - 8 respectively. Selected toe inclinometer data is shown on Figures 9 & 10. Centre line and right hand toe extensometer data is shown on Figures 11 & 12. Selected readings from the hydrostatic settlement gauge are shown on Figure 13.

Filling commenced in the first week of April 1993. Pore pressure response to loading was monitored in terms of B-bar, the ratio between incremental pore pressure rise and increase in vertical stress ($B = \Delta u / \Delta\sigma_1$). Filling was halted for 10 days once the embankment had reached a height of 3.0m to observe pore pressure response and settlement. It had been expected that pore pressure would drop by about 0.3 - 0.5m in this time; almost no change was observed despite settlement of approximately 100mm. Lateral deflection at the toes was approximately 25mm in the peat layer but indicated as pinching-in near original ground level (+1.6m AOD).

A further 2m of fill was placed over the next three weeks. This was followed by a six week pause while the Contractor used the area as a preparation platform for adjacent piling works. Reference to Figure 10 indicates that as in the previous pause, settlement was occurring throughout the foundation although almost no pore pressure dissipationwas being recorded by the piezometers. Surface settlement in that period was 150mm. Seepage was observed from the drainage blanket indicating that the wick drains were functioning; hence there was a suspicion that the piezometers, positioned in an area without wick drains, were not accurately reflecting the pore pressures in the rest of the foundation. Filling with Class 6C material re-commenced and was complete by the end of July. A non-woven geotextile was laid to act as a separator between the brick fill and sand surcharge.

Surcharge was placed throughout August. Prior to placing the final lift, inclinometer readings indicated only 50mm lateral movement, concentrated in the peat. Although B-bar was approaching 1.0, it was felt that this was misleading. Continued settlement and limited lateral deflection suggested that the embankment foundation was capable of taking the final lift of material. Placing of the final lift was substantially complete on Friday 10 September. Dramatic pore pressure changes were observed on the following Monday during final trimming: there was a drop of approximately 1m head at the centreline and comparable rises throughout the rest of the foundation, indicating failure. Heave of 200mm was observed in the toe extensometers. The shape of the hydrostatic settlement gauge plot indicates that the embankment had split and rotated although the toe inclinometer readings still showed no lateral movement at ground level and only minor movement in the peat. Small swallow holes appeared in the surface of the surcharge along with a very faint step along the centreline. A trial pit was excavated into the brick fill however no crack was observed. The following day the four inclinometers were unreadable as were four of five extensometers and the hydrostatic profile gauge. A metre of surcharge was removed over a 150m length in a bid to prevent complete collapse. Over the following weeks, the centreline extensometer kinked and became unreadable. New inclinometers were installed at the toes.

The embankment was left for eight months to settle and gain strength prior to a second attempt to reach the full surcharge height. During this time, piezometers on the left hand side of the embankment began to fail, probably due to pulling of the tubing and consequent breaches of the bentonite seals. However, the toe extensometers could be read intermittently suggesting that the embankment blocks were settling back towards the centreline. It should be noted that the replacement inclinometers did not indicate such movements.

The metre of surcharge was reinstated over a period of four weeks in a series of 250mm lifts. Surcharge was placed over the 25m length of the embankment straddling the instruments. The foundation reaction was observed for two full days. Material was placed over the rest of the embankment if it was felt that there had been no adverse reaction to loading. The last lift induced pore pressure rises equivalent to B-bar = 1.0 and it was decided not to place any additional material. The embankment remained in surcharge for a further four months.

During removal of the surcharge, the geotextile separator placed on the surface of the brick fill was torn, revealing the suspected crack. The crack, located along the centreline of the embankment, was 100m long by 0.6m wide and extended to the base of the fill where clay could be seen. It confirmed that a brittle tensile failure had occurred. Trenches were excavated across the embankment to establish the longitudinal and lateral extent of the crack.

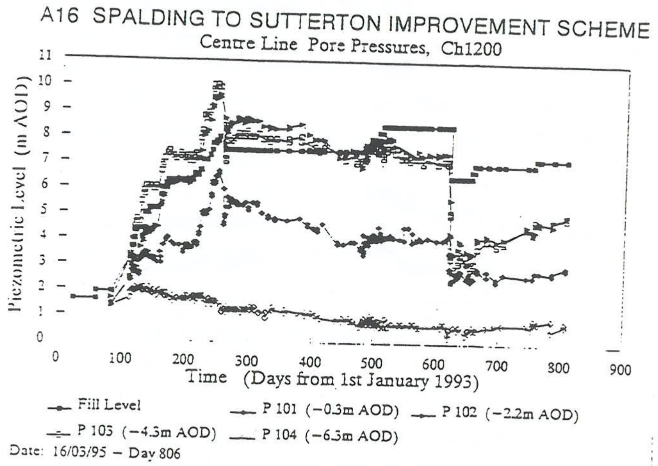

Figure 4: Centre line piezometer readings

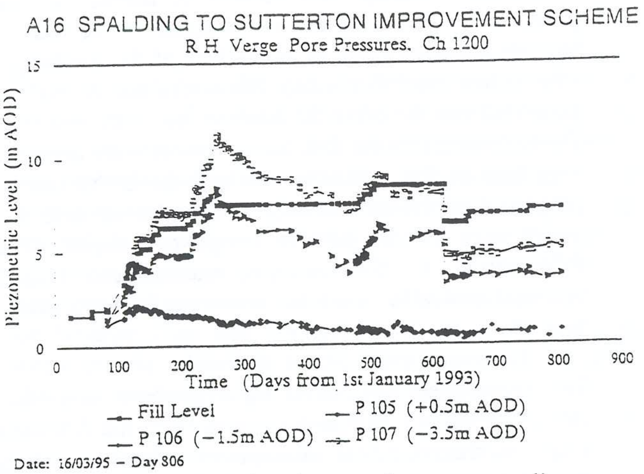

Figure 5: Right hand verge piezometer readings

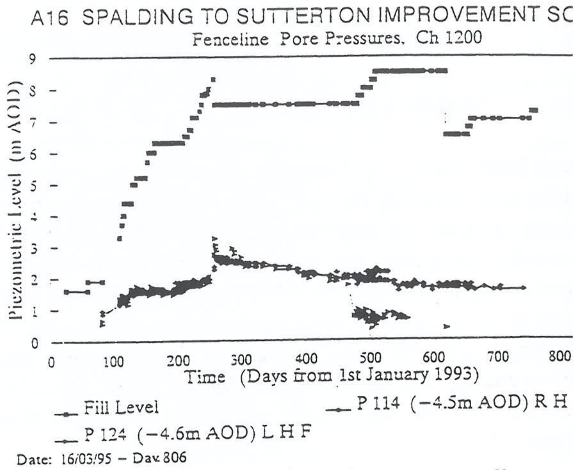

Figure 8: Fence line piezometer readings

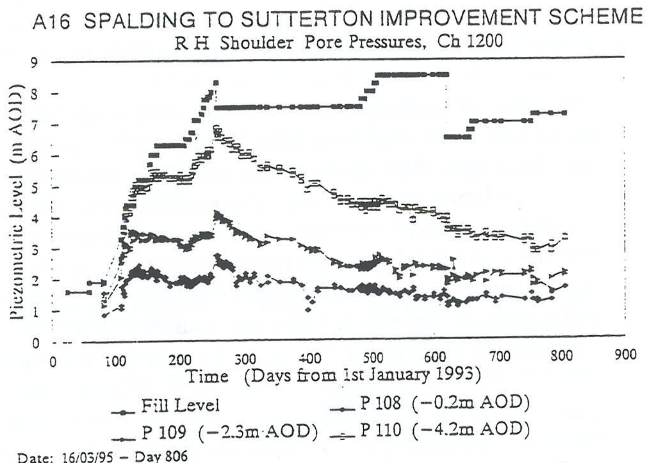

Figure 6: Right hand shoulder piezometer readings

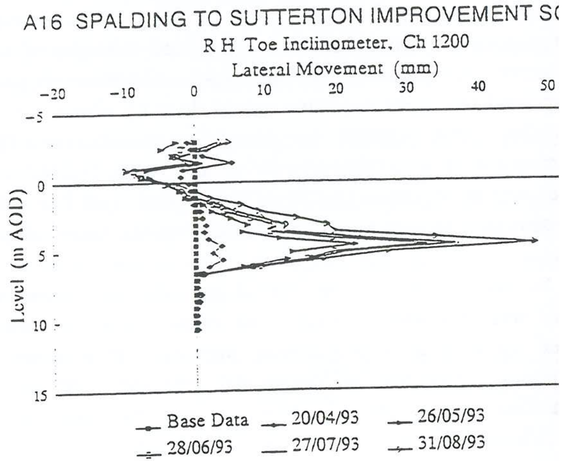

Figure 9: Right hand toe inclinometer reading

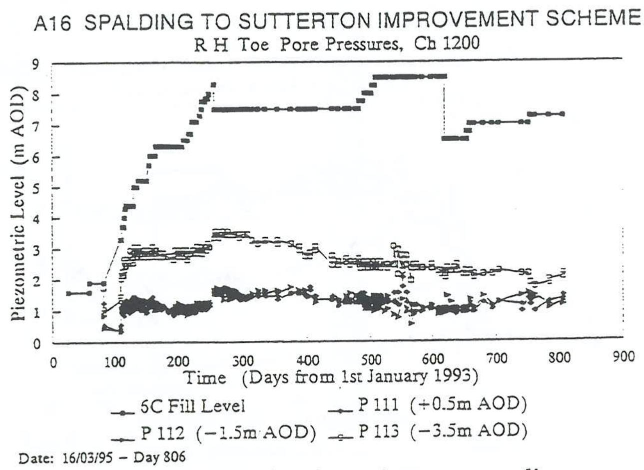

Figure 7: Right hand toe piezometer readings

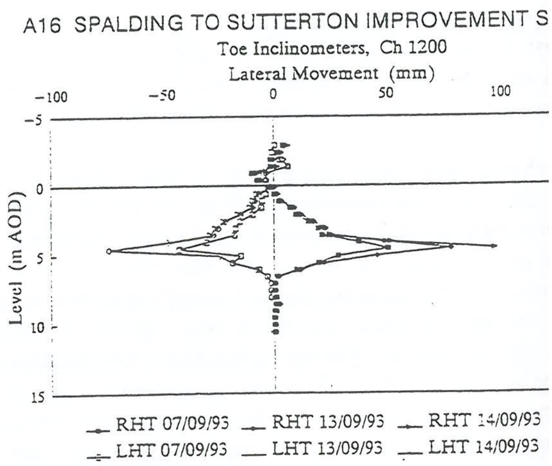

Figure 10: Left and right toe inclinometer readi

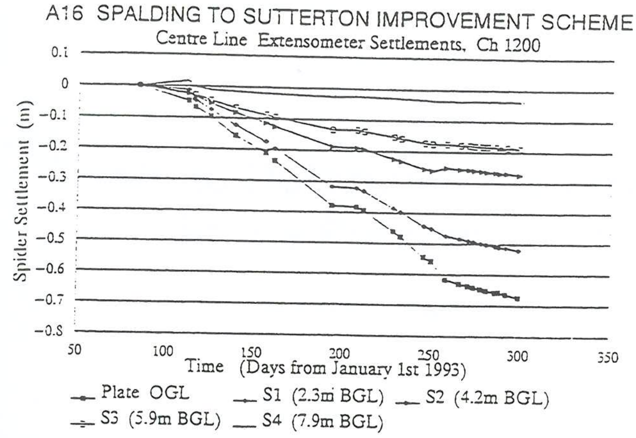

Figure 11: Centre line extensometer readings

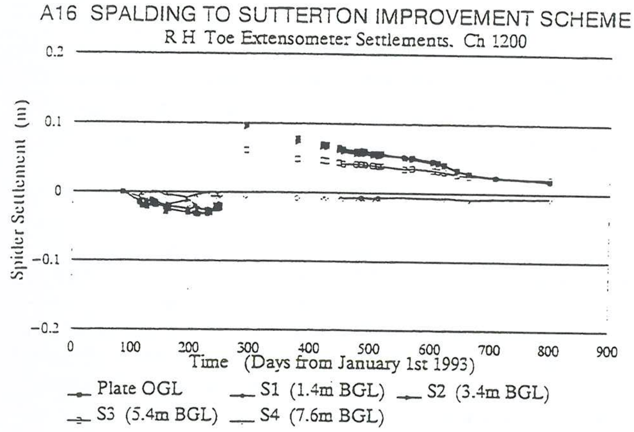

Figure 12: Right hand toe extensometer readings

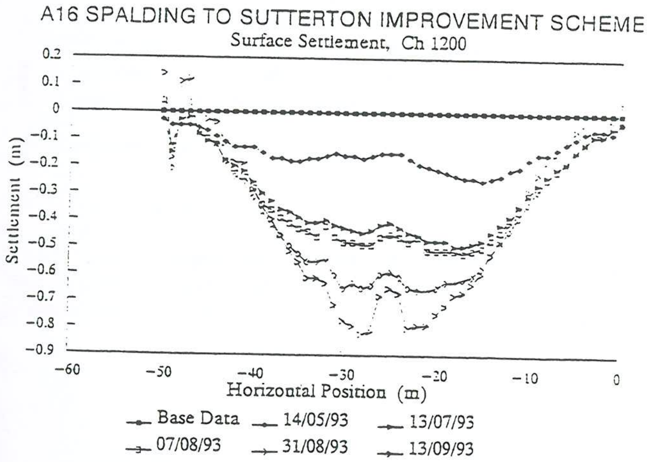

Figure 13: Hydrostatic profile gauge readings

3.2 Remedial Measures

Future stability of the embankment was considered when formulating the remedial measures. Two options were considered: complete removal and reconstruction of the embankment; and grouting of the crack. The choice of solution depended upon the response of the foundation and embankment to the second episode of surcharging.

A review of the data indicated that the embankment had not spread or rotated during the second surcharging. The main evidence for this came from the pore pressure response. During the original failure, sharp rises in pore pressure were observed throughout the foundation from the verge to the fence lines in response to lateral squeezing of the soil; no comparable behaviour was observed during the second episode of surcharging. Readings from the toe extensometers indicated that the blocks had steadily settled 50mm since the original movement. The replacement inclinometers installed at the toes had indicated only slight movement.

On the basis of the above assessment, remedial measures comprised:

- ➢ Grouting of the crack with free-flowing low density foam concrete;
- ➢ heavy compaction of the surface;
- ➢ laying of a woven polyester structural geotextile; and
- ➢ additional transverse steel in the continuously reinforced concrete roadbase.

4 CONCLUSIONS

Instrumentation is vital in controlling construction of embankments on soft clay however it is not without problems. During construction, seemingly contradictory data was being provided by the instruments leading to uncertainty about which ones to believe, although in hindsight all of the warning signs of impending failure were there. Consolidation was occurring, as evidenced by settlement throughout the foundation and seepage of water from the drainage blanket, however there was no concomitant reduction in pore pressure. The lack of pore pressure dissipation suggested that the piezometers were not functioning correctly and therefore greater emphasis was placed on ground movements, in particular inclinometer readings. In hindsight, the piezometers were more reliable indicators of impending failure.

- The non-structural geotextile placed at original ground level, coupled with the brick fill material, created a rigid embankment which suffered a brittle failure.
- Incremental pore pressure response to loading should be taken as the key indicator of foundation stability.
- A lack of ground movement in response to loading should not be used to infer ongoing stability.
- The instrument readings at the time of failure gave enough warning for emergency steps to be taken.
- Removal of the metre of surcharge probably saved the embankment from complete collapse.
- The instrument data enabled design of suitable repairs to the embankment.

5. ACKNOWLEDGEMENTS

Grateful acknowledgment is made to Mott MacDonald Group for permission to write this paper and for assistance during the works described, in particular to Ron Williams and Bill Sim.