Performance of embankments on soft ground: A1033 Hedon Road Improvement Scheme, UK

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ABSTRACT: This paper presents the response of the ground foundation during the construction of the approach embankments to a new flyover at Salt End junction, A1033 Hedon Road Improvement Scheme, UK. The site area was underlain by about 8.0 m depth of soft alluvial deposits and the design specification required, amongst other factors, 100% consolidation of the alluvium under the higher embankments before the construction of the pavement layers could commence. The design solution incorporated ground improvement (prefabricated vertical drains and temporary surcharge) and a basal reinforcement layer in the transition zones next to the bridge abutments. An array of ground instrumentation monitored in real time the pore water pressure and deformation responses of the different strata comprising the ground foundation. The actual values of the primary consolidation parameters were determined from a back analysis of the settlement data.

1 INTRODUCTION

The A1033 Hedon Road Improvement Scheme involved the construction of a new stretch of dual carriageway, 6.7 km in length, that ran alongside the existing two-lane carriageway (west to east alignment) in close proximity to Hull estuary between Mount Pleasant junction, Kingston-upon-Hull, and Salt End junction, Hedon. The main construction on the £45M (2001 data) design and build project commenced in autumn 2001 and was completed by summer 2003. The client was the Highways Agency (UK); the designer Scott Wilson Limited; the main contractor Alfred McAlpine and the client's representative Babtie. The alignment of the new carriageway traversed both greenfield and brownfield areas, which were generally underlain by about 8.0m depth of soft alluvial deposits and with a high groundwater table (within 0.5 m of ground surface).

This paper focuses on the instrumentation and the ground foundation response during the construction of the eastern approach embankment (up to 8.4 m in height) to a new seven-span flyover at Salt End junction. The embankments were reinforced and steepened to 60 degrees on approach to the flyover. The objectives were to limit ongoing settlements to those agreed with the Highways Agency, namely:

1 For the higher embankments, achieve 100% primary consolidation in the alluvial deposits and seek to over-consolidate it to some degree in order to limit secondary compression settlements.
2 Limit the total differential settlements to between 20 and 50 mm over the five-year defect correction period following completion of construction.
3 Restrict ground movements along the line of the embankment toe due to the close proximity of the existing carriageway.

The design solution adopted was the stage construction technique in conjunction with ground improvement (prefabricated vertical drains and temporary surcharge loading). The surcharge would remain in place for a sufficient period in order to reduce the total differential settlements to an acceptable level. For the higher embankments, the surcharge was to induce a degree of over-consolidation in the underlying alluvium.

2 GROUND PROFILE

The Salt End east construction area (Figure 1) was a flat greenfield site (pastureland/playing fields) with the ground surface located about 2.5 m above Ordnance datum (mOD). The embankment site between chainages 6080 and 6400 m was bounded to the north by the existing carriageway and to the south by an open drain.

The ground profile beneath the Salt End east site is shown in Table 1. The very soft alluvium comprised organic clay/silt and firm sandy gravelly clay.

The glacial till layer included gravelly sand lenses (for example, encountered between −9.2 and −9.4 mOD at the location of instruments ME2 and I2, chainage 6120 m (Figure 1)) and was underlain by sandy gravel deposits under artesian pressure.

Table 1. Stratigraphy.

Stratum	Chainage 6120 m Level (mOD)	Chainage 6120 m Thickness (m)	Chainage 6200 m Level (mOD)	Chainage 6200 m Thickness (m)
Ground surface level	+1.5		+1.9	
Crustal alluvium (firm silty clay)	−0.2	1.7	+0.1	1.8
Very soft alluvium	−5.3	5.1	−2.5	2.6
Soft, mottled organic clayey silt	−6.7	1.4	Not encountered	
Firm to very stiff glacial till	−16.2	9.4	−16.0	13.5
Glacial sandy gravel	−32.5	16.3	−32.5	16.5
Highly weathered chalk bedrock			Not proven	

3 EMBANKMENT CONSTRUCTION

After erecting the site boundary and the removal of the top soil layer, a drainage mat comprising a 300-mm deep crushed stone layer sandwiched between Terram 1000 geo-membranes was constructed (Figure 2). With the working platform in place, Coffra Limited installed prefabricated drains (100 mm in width) vertically through the soft ground between chainages 6180 and 6300 m. The site investigation indicated that the thickness of the soft soil deposits varied over the site area. Hence, the drains were inserted through the full depth of the soft soil deposits (using a mandrel advanced under a set driving force) and penetrated up to about 1.0 m into the top of the underlying glacial till layer. Based on the ground model (interpolated from the ground investigation data) and the consolidation parameter values (determined from geotechnical laboratory tests on undisturbed specimens), a network of drains were de-

signed in conjunction with a temporary surcharge to ensure that the geotechnical specification was achieved within the construction programme. The drains were arranged in a triangular grid pattern (plan view) at a centre spacing that varied between 1.0 and 2.0 m along the length of the Salt End site.

A basal reinforcement layer (high-tensile woven geotextile) was also included across the full embankment width in the transition zone to the bridge abutments (founded on pile groups end-bearing in the glacial sandy gravel stratum). The twin aims were to limit differential settlement in the transition along the new carriageway and reduce the lateral movements in the ground beneath the existing carriageway.
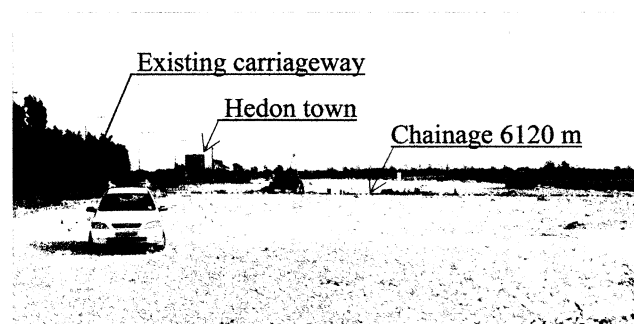
Figure 2. Installing ground instrumentation with drainage mat in place (view eastwards).

4 GROUND INSTRUMENTATION

4.1 Overview

An array of ground instrumentation monitored in real time the ground response during construction. Figure 1 shows the general plan layout of the instrumentation. Figure 3 shows in cross-section one of the two more heavily instrumented sections located at chainages 6120 and 6200 m.
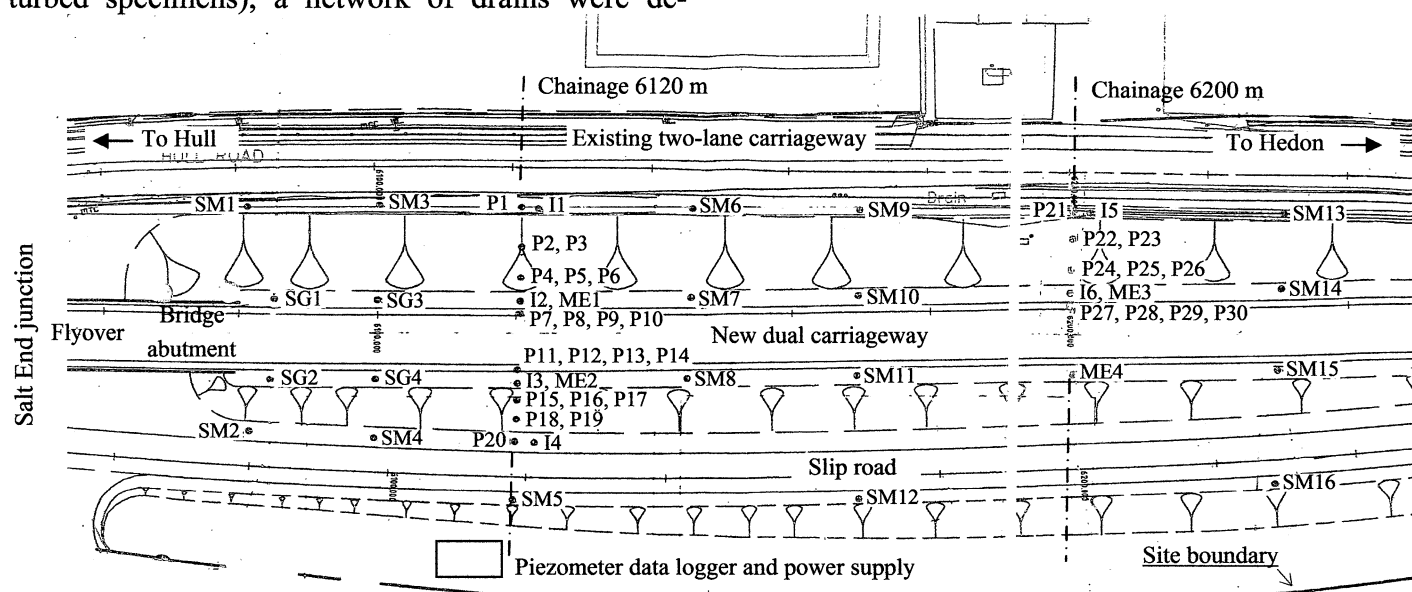
Figure 1. Ground instrumentation at Salt End junction (east). Note: P, piezometer; ME, magnetic extensometer; I, inclinometer; SG, rod and plate settlement gauge; SM, surface settlement marker.
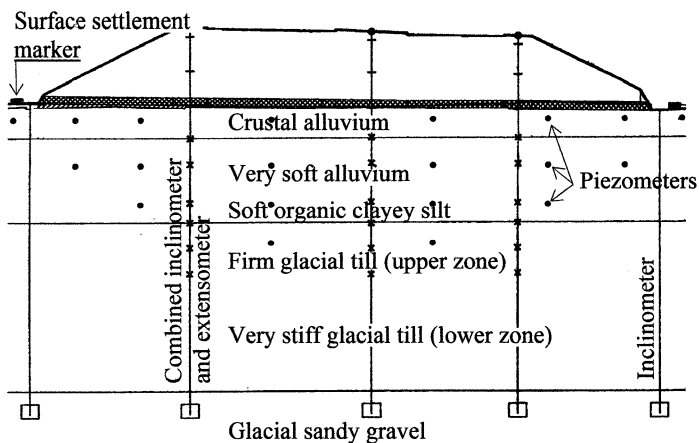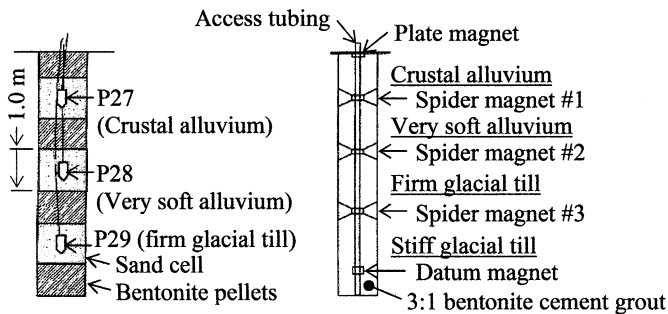
Figure 3. Layout of instrumentation at chainage 6120 m.

(a) Piezometer. (b) Extensometer.
Figure 4. Installation details.

4.2 Pore water pressure

An array of vibrating wire piezometer (P) instruments automatically recorded the pore water pressure response at the mid-height of the strata from which the consolidation responses of the different layers were determined.

Nests of between one and four piezometers were installed in a single borehole, 150 mm in diameter, which had been formed using a light cable-percussive rig (Figures 2, 4a). In plan, the boreholes were located equidistant from the three neighbouring prefabricated drains. The piezometers recorded the pore water pressure over a 1.0 m deep response zone (sand cell) that was centred at the mid-height of the strata. Piezometers were also located remote from the ground foundation to record variations in the natural groundwater level.

The electrical cables from the instruments were connected to an over-ground power supply and data logger unit that were located next to the embankment toe (see Figure 1). The cables were laid in a shallow trench that had been excavated in the drainage mat and backfilled with sand for protection. The site office dialled up the datalogger unit and downloaded the piezometer data in real time to the office PC for analysis.

4.3 Settlement

The settlement response was recorded using an array of magnetic extensometer (ME), rod and plate settlement gauge (SG) and surface settlement marker (SM) instruments (Figure 1). The extensometer boreholes (Figure 4b) were backfilled with 3:1 bentonite-cement grout. The datum magnet was located a distance of 0.5 m above the base of the borehole within the stable ground (glacial till). A series of spider sprung magnets were located at the interfaces between the overlying strata and a plate-mounted magnet was placed above the drainage mat.

The initial instrumentation scheme anticipated locating the datum magnet at depth within the sandy gravel stratum thereby also facilitating measurement of the settlement response over the full thickness of the glacial till layer. However, boiling sand filled the lower section of the open borehole once the borehole installation penetrated the top of the sandy gravel layer (artesian conditions). Hence, the datum magnets were actually located a distance of about 1.0 m above the base of the glacial till layer.

Rows of surface settlement markers (vertical steel pin fixed in a concrete footing) were located along the line of the embankment toe (Figure 1). The steel plates of the rod and plate settlement gauges were in contact with the top of the drainage mat.

4.4 Lateral movement

The lateral movement of the ground foundation was recorded using inclinometer instruments that were located beneath the embankment crest and toe (Figures 1, 3). The bottom section of the inclinometer tube was secured (acting as a reference) by penetrating the top of the glacial till layer by a distance of about 8.0 m. The upper section of the inclinometer tube recorded the lateral movement of the overlying alluvial deposits. Again, the boreholes were backfilled with a 3:1 bentonite-cement grout. In some instances, combined inclinometer and extensometer instruments were installed in the same borehole (for example, beneath the embankment crest, Figure 3).

5 GROUND MONITORING

All of the instrumentation was calibrated and the initial (reference) readings were recorded. The piezometer readings were initially recorded at 10 min intervals over a two-week period to study the variation in the natural groundwater level due to evapotranspiration and tidal action (close proximity to Hull estuary).

The embankment was constructed in a series of short bursts of activity (for practical reasons) and included the use of some locally-available marginal fill material that had been improved by the addition of cement. Different surcharges and periods were adopted along the embankments to fit into the contractor's program requirements.

The piezometers were automatically recorded at one-hour intervals and all of the other instruments

were manually recorded at least twice weekly as the construction progressed. The top of the rod and plate settlement gauge and surface settlement marker instruments were surveyed relative to a topographical benchmark to a measurement accuracy of ± 5.0 mm. The level of the fill material that had been placed above the instrument locations was also recorded following each construction stage. The instruments were shielded from the machine plant inside concrete manhole rings, which were placed above the surrounding ground. The access tubing of the extensometer and inclinometer instruments and the rod sections of the rod and plate settlement gauges were increased in length by the addition of 1.0 m sections as the embankment increased in height. The fill material was then hand compacted around the extended access tubing and rod sections.

The construction rate was controlled from a geotechnical standpoint based on the level of build up in the pore water pressures, the lateral deformations beyond the toe and guarding against a general shear failure. The potential for the latter was assessed from the inclinometer deformations and the toe heave recorded by the surface settlement markers.

The temporary surcharge of uncompacted fill material placed above the final embankment height (Figure 5) was insitu by April 2003. Two metres surcharge depth was placed between chainages 6080 and 6350 m. The surcharge ramped down from 2.0 to 0.0 m in depth between chainages 6350 and 6410 m. A second row of surface settlement markers was installed along the line of the embankment crest at the full surcharge height. Monitoring continued but at a reduced frequency throughout the surcharge period; namely, the piezometers were recorded at three-hour intervals and all of the other instruments were recorded at least once a week.

The surcharge remained in place (typically two to four month period) until such time as a back analysis of the monitoring data (section 7) had proved that the geotechnical design specification had been achieved. The surcharge was then removed. The embankment side slopes were regraded to the finished profile and top soiled after which the construction of the pavement layers could commence.
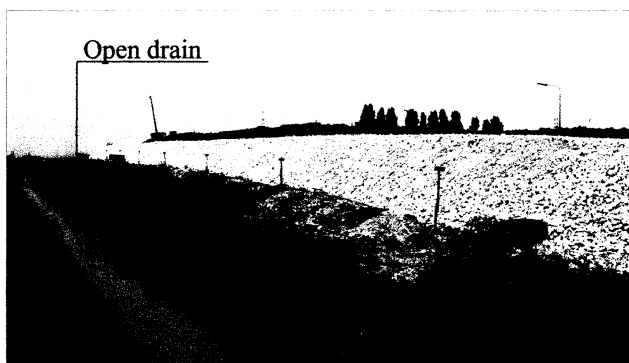

Figure 5. Final embankment height plus temporary surcharge in place (view westwards).

6 GROUND RESPONSE

6.1 Pore water pressure

Figure 6 shows some sample pore water pressure data recorded beneath the north embankment crest at the mid-height of the different strata at chainage 6200 m. The data span over about a three month period leading up to and including the surcharge with about 4.0 m depth of fill already in place. Piezometer P21 in Figure 6 shows the natural groundwater level recorded remote from the stressed ground foundation (typically 1.5 mOD). Initially, the crustal alluvium (P27) consolidated more rapidly (both vertical upward and radial inward flow conditions to the drains) whereas the flow regime in the underlying strata was predominantly radial flow to the drains.

There was an immediate build up in the pore water pressure when the fill material was placed. However, the excess pressures dissipated relatively quickly and the rate of cumulative build up was low. For example, Figure 6 shows that at chainage 6200 m, the excess pore water pressure reached a maximum value of about 25 kPa (2.5-m head of water) for an increase in total vertical stress of about 135 kPa. By the end of a fill stage (1.0 m depth of material placed over a one-day period with consolidation ongoing), the ratio of the step increase in the pore water pressure to the increment in applied stress ranged between 0.25 (crustal alluvium) and 0.35 (alluvium and glacial till layers). At no stage did the recorded pore water pressures reach critical values from a geotechnical stability standpoint.
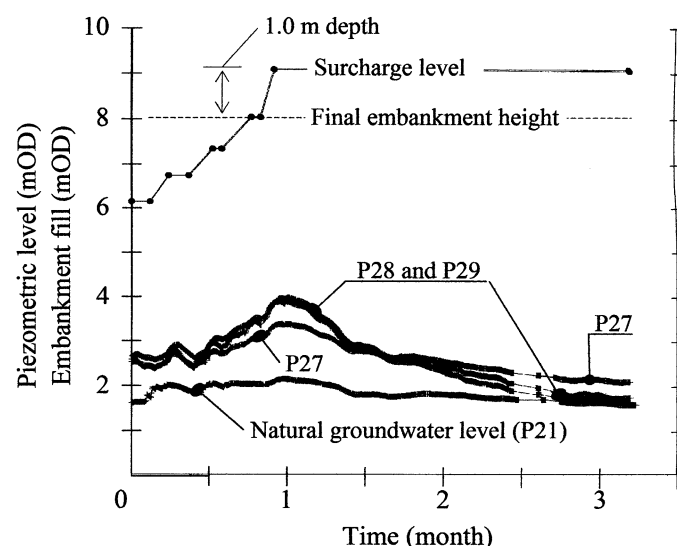

Figure 6. Pore water pressure response at chainage 6200 m.

The excess pore water pressures had largely dissipated by the end of the two-month surcharge period used at chainage 6200 m. Complete dissipation occurred on removing the surcharge indicating that 100% consolidation of the alluvial deposits had been achieved under the final embankment height.

The groundwater level was found to fluctuate periodically with the tides, initially by up to 0.3 m in elevation in the soft alluvial deposits. However, the tidal effect reduced considerably with increasing effective stress (reducing hydraulic conductivity) and became masked by recurrent groundwater level changes arising due to evapo-transpiration (for example, see response of P27 in Figure 6).

6.2 Settlement

Figures 7 and 8 show the settlement response of the different soft soil layers beneath the embankment crest at chainages 6120 and 6200 m. Also included are the step increases in the vertical stress applied to the ground foundation by the embankment.

6.3 Lateral movement

Overall, the deformation response of the ground foundation was largely 1D compression. Figure 9 shows the lateral movements recorded at chainage 6120 m (within the area that included the basal reinforcement layer). Beneath the northern embankment crest and toe (next to existing carriageway in Figures 9b, c), maximum lateral movements of about 40 mm recorded within the soft clayey silt layer. The data from the surface settlement markers located along the embankment toe also indicated no significant heave (all recorded changes in elevation were within measurement accuracy) and there were no obvious signs of distress evident in the bituminous layer. Larger lateral movements of up to 70 mm (Figure 9a) were recorded beneath the side slope nearest the open drain (Figure 5). The maximum movement occurred at a shallower depth (although still within the soft clayey silt layer) and most likely occurred due to the reduction in the shear capacity of the ground foundation caused by the close proximity of the open drain. The lateral deformations of the embankment were also modeled using the finite element method. The recorded deformations were in good agreement with a PLAXIS analysis based on the stiffness values determined from pressuremeter tests.

7 SETTLEMENT BACK ANALYSIS

The actual values of the primary consolidation parameters (Table 2) were determined from a back analysis (curve fitting) of the recorded magnetic extensometer data and the applied load versus time history (Figures 7, 8). The parameter values for the different strata were refined until the theoretical consolidation curve matched the recorded settlement response, assuming 1D compression of the ground foundation.
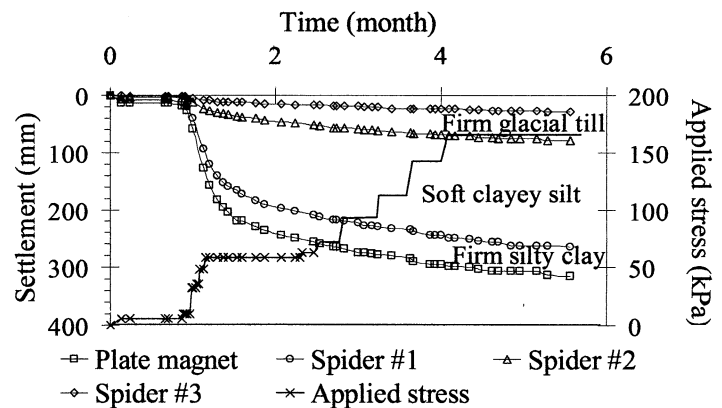

Figure 7. Settlement response from extensometer ME1 at chainage 6120 m.
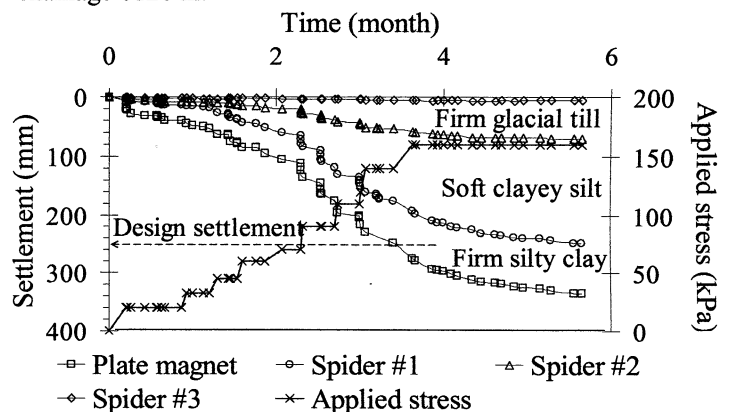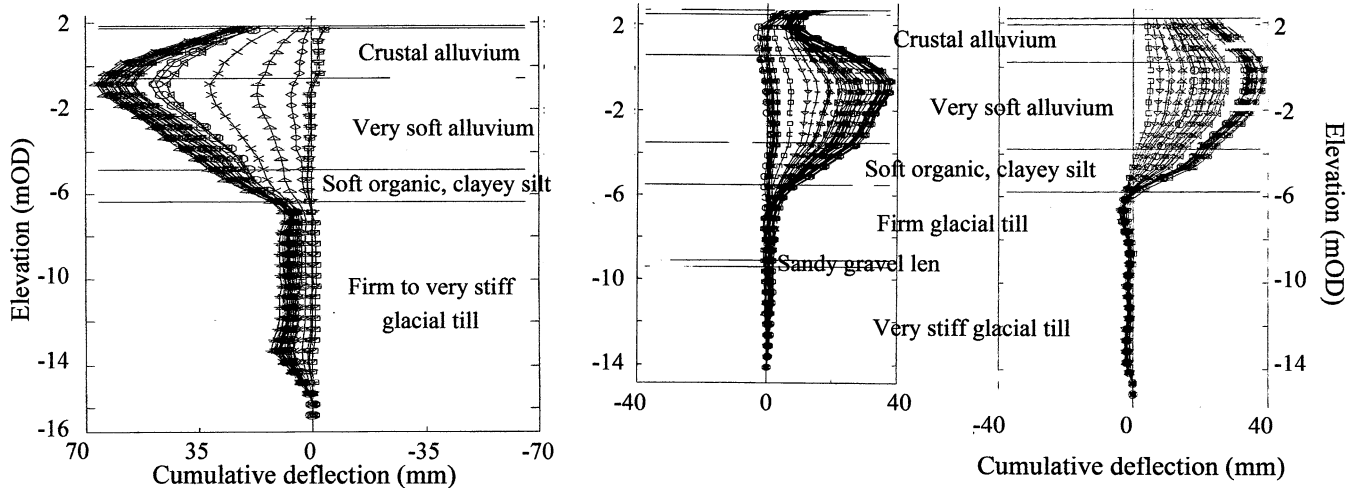

Figure 8. Settlement response from extensometer ME4 at chainage 6200 m

(a) South crest (I3). (b) North crest (I2). (c) North toe (I1).

Figure 9. Lateral ground movements at chainage 6120 m.

Table 2. Consolidation properties determined from settlement back analysis.

Stratum	Coefficient volume change (m^2/MN)	Coefficient of consolidation (m^2/year) Vertical direction Initial	Final	Horizontal direction Initial	Final	Horizontal to vertical coefficient of consolidation ratio
Crustal alluvium	0.30 to 0.50	3.0 to 7.0	1.0 to 2.0	3.0 to 7.0	1.0 to 2.0	1.0 to 1.2
Very soft alluvium	0.40 to 0.70	4.0 to 12	1.0 to 2.0	8.0 to 24	2.0 to 3.0	2.0
Soft, mottled organic clayey silt	0.40 to 0.60	3.0 to 4.0	1.0 to 2.0	6.0 to 7.0	2.0 to 3.0	1.7 to 2.0
Glacial till (firm upper zone)*	0.05 to 0.15	3.0	2.5	3.0	2.5	1.0
(stiff lower zone)	0.01 to 0.05	3.0	—	3.0	—	—

* Penetrated over full depth by vertical drains and included sand lenses of up to 0.2 m in thickness.

Each layer was taken in sequence, commencing with the lowermost layer (glacial till) and using the laboratory-measured parameter values as a starting point. The computations were performed in Microsoft Excel taking into account the vertical drain arrays (1.0 and 2.0 m centre spacings at chainages 6120 and 6200 m, respectively) and the permeability anisotropy of the soft ground deposits. Table 2 lists the initial (greenfield) and final (surcharge) values of the coefficient of primary consolidation that were determined for the vertical and horizontal directions.

The back-analysed values were compared with the laboratory-measured values. Overall, both sets of coefficient of volume change values were in good agreement. However, the back-analysed coefficient of consolidation values were between two and four times greater than the values determined from standard oedometer test data with the discrepancy most likely arising due to scale effects. Moreover, the soft soil layers were cross-anisotropic with consolidation occurring up to twice as fast for horizontal (radial) flow rather than for vertical flow conditions.

The secondary compression index (C_α) values were estimated for the strata using the relationship: $C_\alpha = 0.04C_c$, where C_c is the compression index. For the calculation of the secondary compression settlements, $C_\alpha/(1 + e)$ values of between 0.0035 and 0.0090 were used (where e is the void ratio).

The target settlements for the different strata under the final embankment height (achieving 100% consolidation of the alluvium) were calculated using the back-analysed parameter values in Table 2. The recorded settlements for each layer exceeded the target values proving that the geotechnical design specification had been achieved.

Table 3. Settlement response at chainage 6200 m.

Stratum	Final settlement recorded by ME4 (mm)	Target consolidation settlement (mm)
Crustal alluvium	86	65
Very soft alluvium	176	132
Firm to very (upper)	67	50
stiff glacial till (lower)	6	5
Total settlement (mm)	335	251

For example, Table 3 compares the settlements recorded by extensometer ME4 at the end of the sur-

charge period (applied stress of 160 kPa) with the target values for the final embankment height (applied stress of 120kPa).

The soft soil deposits were lightly over-consolidated (over consolidation ratio value of about 1.3) on removing the surcharge material. Settlements to date following construction of the embankments are in accordance with the design specification.

8 SUMMARY AND CONCLUSIONS

An array of ground instrumentation monitored the pore water pressure and deformation responses of the ground foundation (about 8.0 m in depth) during the construction of the approach embankments.

The degree of consolidation was assessed from the excess pore water pressure values recorded by the vibrating wire piezometers which were located at the mid-height of the different strata. The inclinometer data indicated that the ground foundation, aided by the basal reinforcement to the embankment, essentially deformed in 1D compression.

The actual values of the primary consolidation and secondary compression parameters for the different strata were determined from a back analysis of the magnetic extensometer data.

After a two-month surcharge period, the recorded settlements for the different strata exceeded the target values (100% consolidation of alluvium) set by the design specification for the final loading.

The back-analysed and laboratory-measured coefficient of volume change values were in good agreement. However, the actual coefficient of primary consolidation values were between two and four times greater than the values determined from standard oedometer test data, most likely due to scale effects. The soft soil layers were cross-anisotropic with consolidation occurring up to twice as fast for horizontal flow rather than for vertical flow conditions.

ACKNOWLEDGEMENTS