

Cite this paper as follows:

O'Kelly B.C., Ward P.N. and Raybould M.J. 2008. Slope stabilization of an embankment on the West Coast Main Line, UK. *Proceedings, First International Conference on Transportation Geotechnics, 25th–27th August, Nottingham, UK*. Vol. 1, pp. 335 – 341.

Slope stabilization of an embankment on the West Coast Main Line, UK

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ABSTRACT: A site inspection of an embankment on the West Coast Main Line (UK) revealed a general deterioration of the embankment, significant movement of the trackside services and waterlogging and bulging along the embankment toe. The medium-height embankment, with side slopes at about 30 degrees to the horizontal, was constructed on natural sidelong ground and the site had been an area of previous instability (former ash and slag tip). Site investigation, ground monitoring and analytical modelling indicated that the embankment was at limiting equilibrium and that recent progressive movement was occurring along a rotational slip surface located within the embankment core (ash and slag materials overlying reworked glacial till). The upgrade works comprised the installation of a row of stabilizing piles along the mid-height of the embankment side slope (facilitating Green Zone working); regrading to prevent shallower slips near the embankment crest and the installation of a toe drain that discharged to a nearby stream.

1 INTRODUCTION

The site area is part of an embankment (Figure 1) located at Wreay (about 5 km south-southeast of Carlisle, Cumbria) along a north-south aligned section of the Lancaster to Carlisle railway line (West Coast Mainline, UK) at Ordnance Survey national

grid reference NY 439 482. The Lancaster to Carlisle line was constructed during the mid 1840s and currently comprises two tracks (Up-line and Down-line) supplied with 25 kV overhead electrification. Figure 2 shows some pertinent rail terminology as it relates to this particular site.

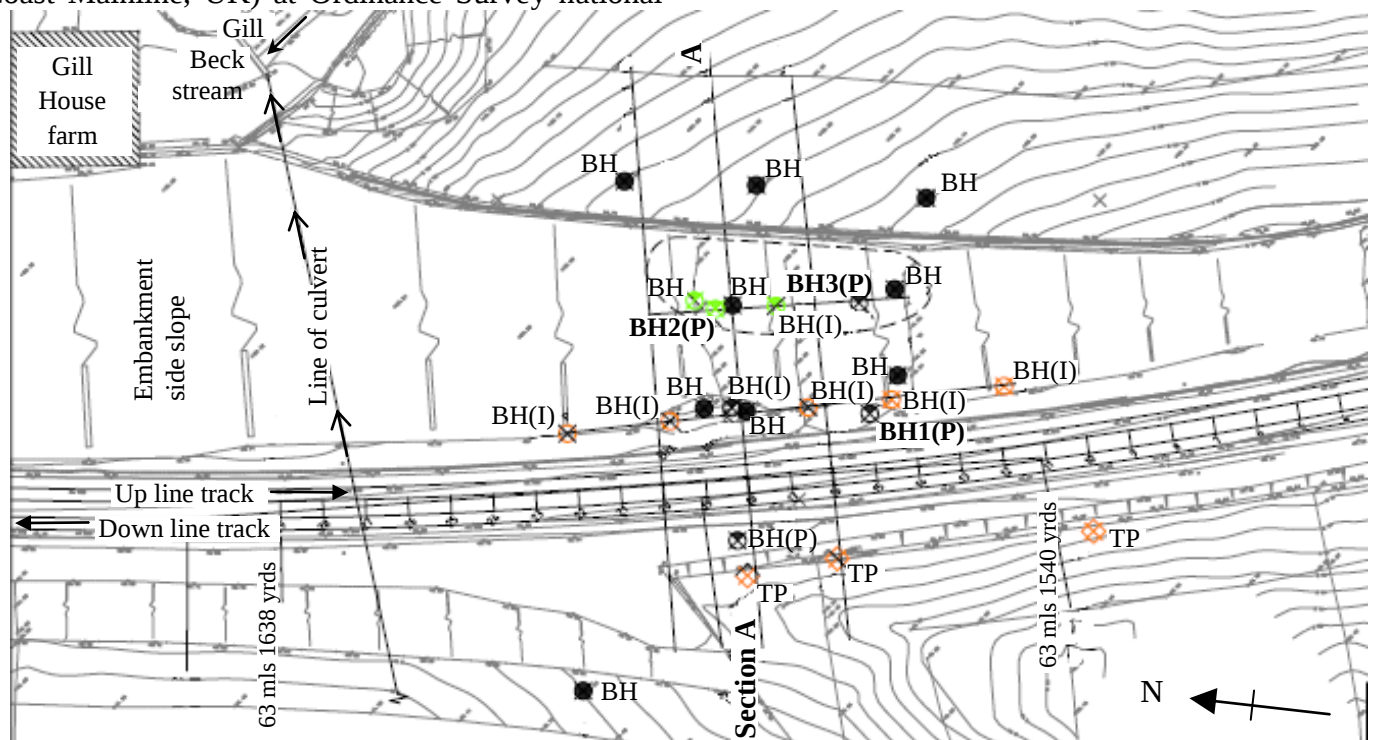


Figure 1. Site location plan. Note: BH, borehole; TP, trial pit; (I), includes inclinometer; (P), includes piezometer.

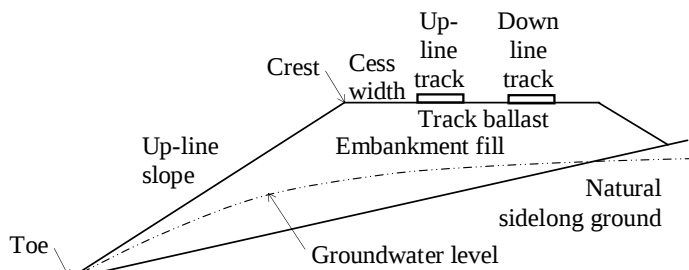


Figure 2. Terminology associated with twin-track railway.

The embankment has an overall length of about 300 m and had been constructed on natural sidelong ground, which generally sloped in a north eastward direction. The embankment slope on the Up-line side was up to 13 m in height whereas the side slope on the Down-line side was less than 4 m in height (typically between 2 and 3 m). The side slopes were densely vegetated with grass, shrubs and mixed wood (Figure 3).



Figure 3. View southwards along Up-line slope.

Two previous slope failures were known to have occurred on the Up-line side, adjacent to the site area behind Gill House farm (Figure 1).

Network Rail instructed Edmund Nuttall Limited to design and construct the necessary upgrade works, which were instigated following a site inspection (revealed leaning gantry at 63 mls 1556 yds, waterlogged ground conditions and bulging along the Up-line toe) and track monitoring data (indicated adverse movement of the rail tracks between 63 mls 1534 yds and 63 mls 1610 yds). Note that

distances and locations along the rail lines are traditionally given in miles and yards in the industry (one mile equals 1.609 km and one yard equals 0.914 m).

Scott Wilson Limited was appointed by Edmund Nuttall Limited to undertake a desk study, site inspections, ground investigation and to design the necessary upgrade works to reduce future movement of the track and improve the stability of the critical embankment section identified. This paper presents the geotechnical design and construction of the upgrade works.

2 GROUND CONDITIONS

2.1 Ground investigation

The ground investigation works were detailed by Scott Wilson and carried out in stages by Ritchies Limited (UK) between April and October 2001. The works comprised 21 cable percussive boreholes, three trial pits and the installation of three standpipe-piezometer and seven inclinometer instruments. The locations of the exploratory holes and ground instrumentation are shown in Figure 1. Table 1 lists the inferred sequence of the different strata encountered. Figure 4 shows a typical geological cross-section for the site area. The undrained shear strength profile with depth was determined using empirical correlations from the N blow count values (Figure 5) that were measured from Standard Penetration tests (SPT) in the exploratory boreholes.

Table 1. Stratigraphy beneath embankment centreline.

Stratum	Description	Thickness (m)
Ballast		0.3 to 0.5
Made ground	Loose to medium-dense ash, slag and cinders	3.8 to 4.4
Embankment fill	Very soft to firm silty sandy clay with some loose silty clayey sand	2.7 to 6.4
Glacial till	Firm to stiff glacial till	Not proven
	with fine sand lens (ground foundation)	

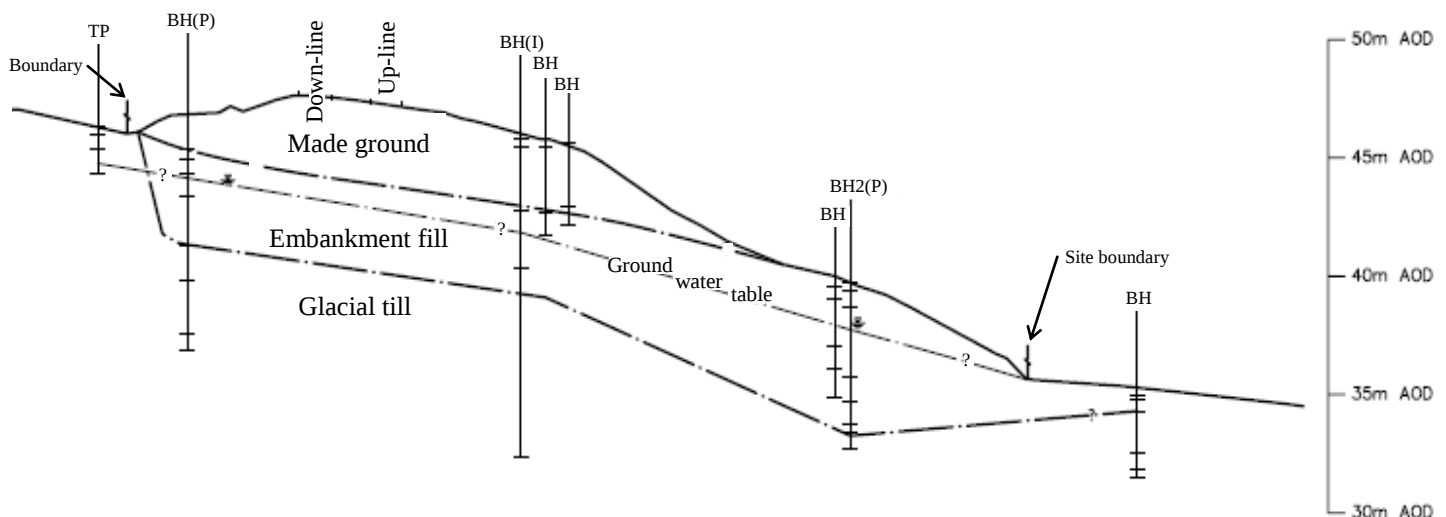


Figure 4. Geological profile at section AA in Figure 1. Note: BH, borehole; TP, trial pit; (P), includes piezometer.

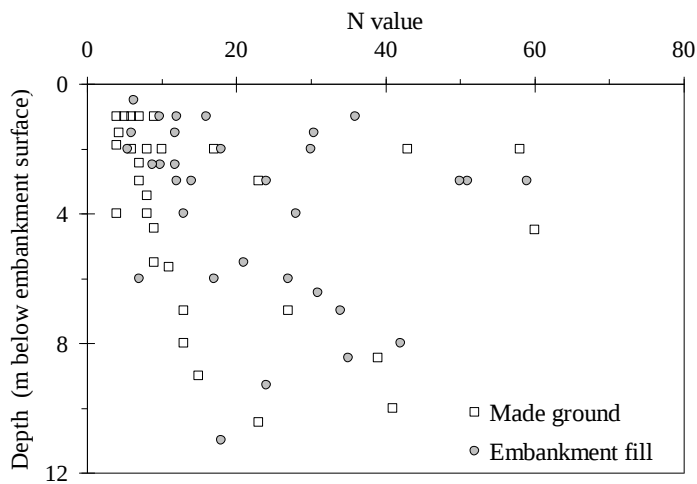


Figure 5. Standard Penetration test data.

The Network Rail Hazard directory (issue 3rd January 2002) listed the site area as a former railway tip (Wreay ash tip), which explained the significant depths of ash and slag materials that constituted the upper section of the embankment. Tipping began on the Down-line side in the 1900s with the level of the ash reaching track level by around 1914. Much of the ash material has since been removed from the Downside field for use in the manufacture of breezeblocks.

The lower section of the embankment was most likely constructed using locally-sourced glacial till material. The soils are generally of low plasticity and the SPT N-blow values (Figure 5) indicated frequent loose zones present.

The British Geological Survey map (Penrith: Sheet 24, 1:50,000 scale) and Geological Memoir (Appleby, Ulleswater and Haweswater) indicated that the solid geology at the site is Penrith Sandstone (poorly silicified red-brown sandstone).

2.2 Groundwater

The river Petterill is located about 200 m to the northeast of the site. Gill Beck, one of its tributaries, is culverted beneath the north end of the site area and exits at the embankment toe on the Up-line side, just south of Gill House farm (Figure 1).

Site inspections revealed seepage, water logging and bulging along the Up-line toe. However, there was no evidence of gullies or pre-wash slope failure.

The piezometer and inclinometer instruments were located along the Up-cess and at the mid-height of the embankment slope within the length of the toe bulge area. The response zone (sand cell) of the three piezometers at borehole locations BH1(P), BH2(P) and BH3(P) (Figure 1) were over the full depth of the embankment fill material and the instruments were periodically monitored between April 2001 and May 2002.

The data shown in Figure 6 indicated a relatively high groundwater table that fluctuated seasonally

(substantial rise in groundwater levels between September and October 2001). The groundwater table was generally located between 3.7 and 6.0 m below the embankment crest level (within the reworked glacial till material) and between 0.3 and 2.0 m below the mid-height of the slope face. The groundwater table was almost coincident with the ground surface at the embankment toe. However, no seepage had been observed from the slope face itself.

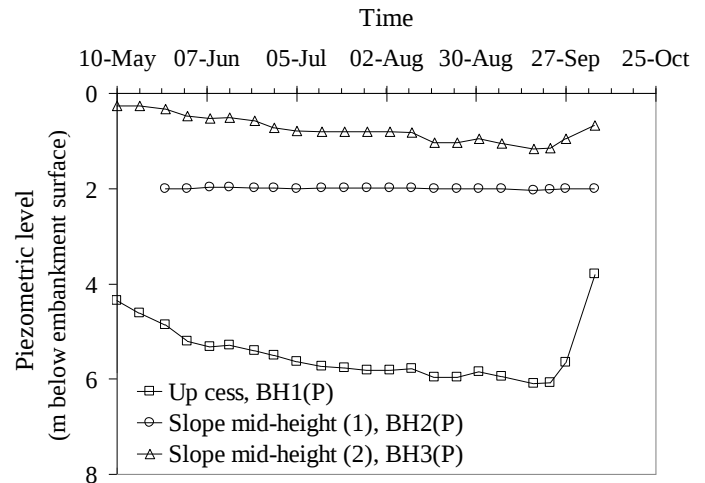


Figure 6. Groundwater levels within the embankment core.

2.3 Ground movement

Two inclinometers were initially installed to detect ground movement. Another five inclinometers were later installed to measure and define the extent of that movement. The lower section of the inclinometer tubes, 50 mm in diameter, were anchored to a depth of at least 3.0 m in the underlying glacial till foundation.

The instruments indicated that the embankment was moving laterally along a shear zone located at a depth of between 2.0 and 5.0 m below track level (within made ground and embankment fill layers) with up to 30 mm of movement recorded between April and September 2001. Monitoring of the track distortion including the Up-line and Down-line rail levels, cants and twists, and the Six-foot widths (refer to Figure 2) along with 22 target-monitoring points was undertaken on a weekly basis from 63 mls 1368 yds to 63 mls 1720 yds between June 2001 and May 2002.

A settlement trough, which affected both the Up and Down-line tracks over the full length of the site area, was identified (up to 16 mm of vertical settlement recorded) although the degree of track distortion was still within acceptable limits. Level monitoring of two gantry stanchions located in the Up-cess indicated that settlement was ongoing with up to 12 mm settlement recorded between July and September 2001. Moreover, bulging of the Up-line toe and tilting of the existing boundary fence, which had been installed about four years previously, had occurred over a distance of about 60 m.

3 GROUND MODEL AND DESIGN PARAMETERS

The existing embankment profile was accurately determined from a topographical survey (5.0 m square grid) undertaken by Interactive Track Services Limited (UK) in June 2001. Along the Up-line side, the embankment ranged between 9.5 and 13.0 m in height, with the Up-line side sloping at an overall gradient of about one vertical to 1.7 horizontal (30 degrees). The ground model (typical section presented earlier in Figure 4) was based on the exploratory borehole data and the recorded groundwater levels (Figure 6).

From the site inspection, ground conditions and a review of the monitoring data, it was concluded that the upper embankment section was slowly but progressively moving along a rotational slip surface located within the embankment core. The gantry at 63 mls 1556 yds was most likely leaning due to a shallower rotational slip which had induced a bearing capacity type failure.

A slope stability back-analysis was carried out on the existing embankment profile using the Geosolve *SLOPE W* program and assuming a limiting equilibrium condition (factor of safety (FOS) value of unity) in order to determine the mobilized values of the effective stress shear strength parameters for the different strata (Table 2). The analysis was carried out using the Morgenstern and Price Method of Slices in *SLOPE W*. Data from ring shear (peak), shearbox and consolidated-undrained triaxial compression tests with pore water pressure measurement (sets of three 38 mm diameter test specimens) were used as the initial input values.

A sensitivity analysis considered the FOS values mobilized for the most probable and most unfavourable ground conditions and the effects of variations in the groundwater levels. High groundwater levels are unconservative for the purpose of back calculating the values of the shear strength parameters. It was concluded that the critical slip surface, which

ran from the Up-line cess and day-lighted at the embankment toe, was activated during transient rises in the groundwater level within the embankment fill material during torrential rainstorm events.

Table 2. Design values from *SLOPE W* back analysis.

Stratum	Bulk unit weight (kN/m ³)	Effective cohesion c' (kPa)	Effective friction angle, ϕ' (degree)
Made ground	18	0	27
Embankment fill	20	3	22
Glacial till	21	3	32

4 UPGRADE WORKS

4.1 General

The upgrade works comprised the construction of an embedded pile retaining wall (shear dowels) along the mid-height of the Up-line embankment slope, the re-grading of the slope face and the construction of a toe drain (Figure 7). The embankment slope was regraded to one vertical in 2.5 horizontal above the new retaining wall and to one vertical in 2.0 horizontal below. In addition, a 2.8 m wide cess walkway and shoulder were constructed along the Up-line. The upgrade works used best practice measures, through a process of value engineering, to reduce the hazard of further slope movement that would affect the rail track.

A row of reinforced-concrete piles (46 number in total) were installed through the full depth of the embankment and penetrated the underlying stable ground, thereby preventing further lateral slope movement and reducing future movements at the track-bed level. The location of the piles at the slope mid-height was the most efficient and caused the least disruption to rail operations (clear spacing of 17.2 m between the pile row and the nearest Up-line rail). The re-grading of the upper slope reduced the

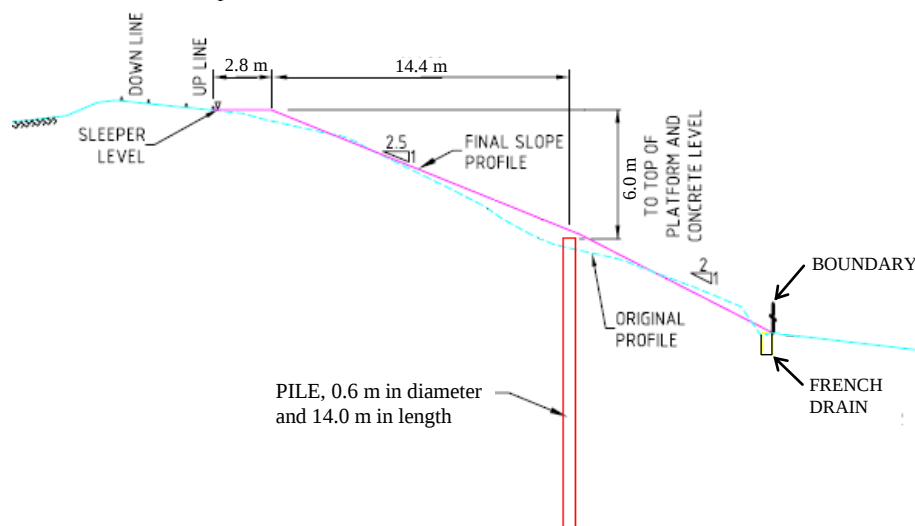


Figure 7. Upgraded embankment.

risk of shallower rotational slips, which had occurred near the embankment crest, and the possibility of lateral spreading of the track-bed material. The construction of the toe drain with outfall to Gill Beck stream reduced the high groundwater table/water-logging near the embankment toe.

A number of alternative upgrade solutions were considered (but ultimately rejected) as follows:

- Construction of a pile retaining wall located along the embankment crest. Rejected as works would have had to be carried out next to the overhead line electrification; restricted sighting distance and would have also required possessions and larger (diameter and length) piles. Possession of all or part of the rail track was not possible during the course of the upgrade works.
- Regrading the Up-line slope with a toe retaining-structure in place. The embedment requirement for the toe retaining structure led to concerns that excessive ground movement would have occurred during excavation works along the embankment toe;
- Regrading works over the full Up-line slope would have required land purchase next to the embankment toe with possible delays in reaching agreement for permanent land take;
- Counterfort drainage was rejected over concerns that excessive settlement would occur during the excavation works for the counterfort drains.

4.2 Design

The upgrade works were designed to improve the FOS value against slope instability as follows:

- The FOS value against a reactivation of the critical slip surface was increased from unity to 1.3;
- The FOS value against shallower secondary slips occurring within the upper section of the regraded slope was increased to at least 1.3.

As well as providing the necessary additional horizontal resistance to improve the global slope stability, the piles were also designed to support the

upper section of the embankment in the event of a slip failure occurring below the location of the pile embedded retaining wall.

A *SLOPE W* analysis using the effective stress strength parameter values in Table 2, and with a vertical stress of 50 kPa applied along the twin tracks, indicated that a row of piles capable of exerting a horizontal resistance of 200 kN/m run at the mid-height of the embankment slope would increase the FOS value against a reactivation of the critical slip surface from unity to 1.3. The analysis also indicated that all potential shallower slips that day-lighted just upslope of the retaining wall would have FOS values greater than 1.3.

The pile design was based on a plane-strain analysis of the retaining wall using *WALLAP* geotechnical software assuming a loss of passive support of up to 1.0 m in depth in the event of a slip failure occurring further down the embankment slope. Modified coefficient of earth pressure values were used to accurately represent the sloping ground. A single row of cantilevered, reinforced concrete piles (600 mm in diameter and between 12.4 and 14.0 m in length) was capable of providing the necessary horizontal resistance of 200 kN/m run. The piles were spaced at 1.5 m centres along the row. The glacial till foundation provided the lateral resistance to the piles (FOS value of 2.0 against a rotation failure).

The steel reinforcement (10 number T40 mm diameter main steel bars with a 75 mm cover) was designed in accordance with BS5400 to provide a structural section capable of resisting the system of working loads and bending moments (applying a FOS value of 1.65) and for a design life of 120 years. The design calculations indicated that the bending moments and shear forces reduced to zero at a distance of about 13 m below the pile head level. The geometrical arrangement of the piles (46-number at 1.5 m centre spacing) also permitted the easy migration of groundwater through the embedded wall.

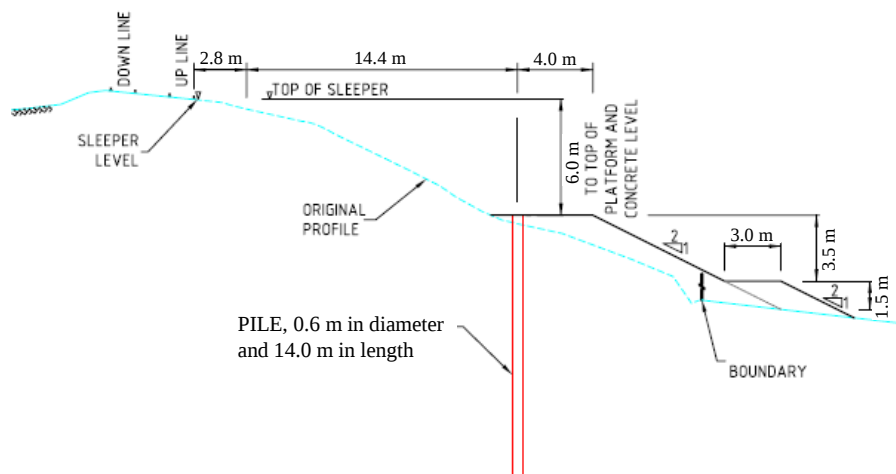


Figure 8. Temporary works.

4.3 Construction

The engineering works were carried out between March and April 2002 and at no stage was temporary possession of the rail track necessary (works sufficiently far away for Green Zone working).

A temporary haul road was constructed to access the base of the slope, which had been cleared of all vegetation. Then a piling platform up to 5.0 m in width was constructed along the length of the upgrade works (70 m in overall length between 63 mls 1534 yds and 63 mls 1610 yds) at the mid-height of the embankment slope (Figure 8). The platform was constructed using granular fill, progressively benching into the lower embankment slope to reach a distance of 1.3 m above the pile cut-off level. The benches were excavated in bays (typically 5.0 m in length, 600 mm in height and with a slope angle of about 60° to the horizontal) parallel to the track.

The bored piles were installed through the platform by Cementation Skanska using the continuous flight auger technique. The design included a slope stability analysis for the set up with the piling rig working above the platform with berm (FOS value on slope instability of 1.2). The spoil from the base of the boreholes indicated that the piles had penetrated the full depth of the glacial till layer and were founded in the underlying glacial sands. The piles were cast using 40 N concrete (28 day strength). After hardening, the pile heads were trimmed back to sound concrete. Integrity testing indicated that the integrity of all of the piles was sound.

The upper section of the embankment slope was regraded to the finished profile (one vertical to 2.5 horizontal). The unsuitable embankment fill was removed from the slope face in benches and replaced with compacted Highways Agency specification Class 6B granular fill material. The slope face was covered with a 300 mm deep Class 6F2 capping layer.

The piling platform was subsequently removed followed by regrading of the lower section of the embankment slope to the finished profile (one vertical to 2.0 horizontal). A drainage layer (2.0 m in width and 0.5 m in depth), which was wrapped in a geotextile membrane, was constructed at the interface between the ash/slag layer and the underlying embankment fill. The drainage layer was necessary since the Class 6F2 material (well graded limestone, three inch size down to dust) that had been used in the capping layer tends to hydrate when moist creating a very low permeable surface. A French drain was constructed along the line of the boundary fence near to the embankment toe with an outfall to a headwall structure at Gill Beck stream. The drain (0.5 m in width and 1.0 m in depth) was excavated in short lengths and backfilled without delay with

Highways Agency specification Type B filter material. Finally, geo-matting and seeding were placed over the entire embankment slope to provide erosion protection and for the rapid reestablishment of vegetation.

5 POST CONSTRUCTION

Periodic monitoring of the track along the length of the site continued until June 2002 (three months after the substantial completion of the works). The data indicated that negligible movement had occurred at track-bed level during the construction works and the following three-month period.

A six-month review of the site in December 2002 indicated no discernable deficiencies or defects in the tracks. Interestingly, all of the inclinometers located above the embedded retaining wall had recorded ground movements of up to 10 mm up the slope, towards the track. It is postulated that the ground movements up the slope that had been recorded by the inclinometers were as a result of the stress relief that occurred following the removal of the pile platform and the slope regrading works.

6 SUMMARY

The ground investigation, monitoring and numerical analysis indicated that the upper section of the embankment was slowly but progressively moving (limiting equilibrium) along a rotational slip surface located within the embankment core (ash and slag materials overlying reworked glacial till). The groundwater level within the embankment core was relatively high due to the site topography (steeply sloping sidelong ground). The upgrade works comprised the construction of a row of stabilising piles (shear dowels) at the mid-height of the embankment slope, regrading of the slope face to prevent secondary shallow slips near the embankment crest and the construction of a toe drain.

The effective stress shear strength parameters for the design were determined from a slope stability back-analysis of the critical embankment section which was at limiting equilibrium. The critical slip surface, which ran from the Up-line cess and daylighted at the embankment toe, was activated by transient rises in the groundwater table within the embankment core during torrential rainstorm events.

The 600-mm diameter stabilising piles, which provided the necessary horizontal resistance to increase the FOS value against slope instability from unity to at least 1.3, had the desired effect of preventing lateral movement of the embankment core, thereby reduced future movement at track bed level to an acceptable amount.