Soil nail remediation of seasonal slope movements in an old clay fill embankment dam

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Abstract

A curious slope movement pattern has been recorded during 10 years of monitoring at one unstable section of the downstream slope of Aldenham embankment dam, in Hertfordshire, UK. A slip plane in the upper slope was reactivated each summer, which was caused by shrinkage of the tree-covered toe of the embankment due to soil moisture deficit, leading to loss of support to and reactivation of the grassed upper slope slip mass. This led to a seasonal, ratcheting accumulation of crest settlement – a type of mechanism that presents a continual serviceability problem to infrastructure embankments as well as dams.

A soil nailing scheme was designed and installed to support the upper slope slip mass, and then grassed over. Inclinometer and wave wall settlement data in the seven years following nail installation showed the scheme to be successful, even resulting in some upslope recovery movements.

Figure 1: Location map

Introduction

In the late 18th and early 19th centuries, many embankment dams were built in the UK for landscaping purposes and for canal reservoirs. At least about 200 of them are still in use today (Kennard, 1975). These early dams were constructed in a similar fashion to the railway embankments of southern England in the mid-19th century onwards – from thick uncompacted layers of dumped clay fill (Vaughan et al, 2004) – and have suffered the same instability problems.

Vegetated clay soils in temperate climates are subjected to seasonal cycles of pore water pressures as high suctions during peak water demand by vegetation in relatively dry summer months dissipate as water demand lowers and rainfall increases in the winter months. In clay slopes, the resulting cycles of shrinkage and swelling of the ground surface leads to a net ratcheting downward and outward movement of the slope (Russell et al, 2000).

Furthermore, the net movements develop on preferred surfaces where strain softening is occurring, starting from the toe of the slope, and, after sufficient number of pore water pressure cycles, can lead to progressive failure (Nyambayo et al, 2004; Take and Bolton, 2011; Vaughan et al, 2004).

The higher water demand and extensive root systems of trees increase shrinkage and swelling cycles (Biddle, 1998; Driscoll, 1983), and some trees (eg oak) have a higher water demand than others (eg elder) (Biddle, 1998). While trees can improve slope stability, it is increasingly recognised that they can cause summertime settlements of clay-fill infrastructure embankments that do not fully recover in the winter (Andrei, 2000; Ridley et al, 2004; Russell et al, 2000). Many of the early embankment dams are tree covered (Kennard, 1975; Hoskins and Rice, 1992) but, to date, no monitoring data has been available to assess the influence of trees on any settlement.

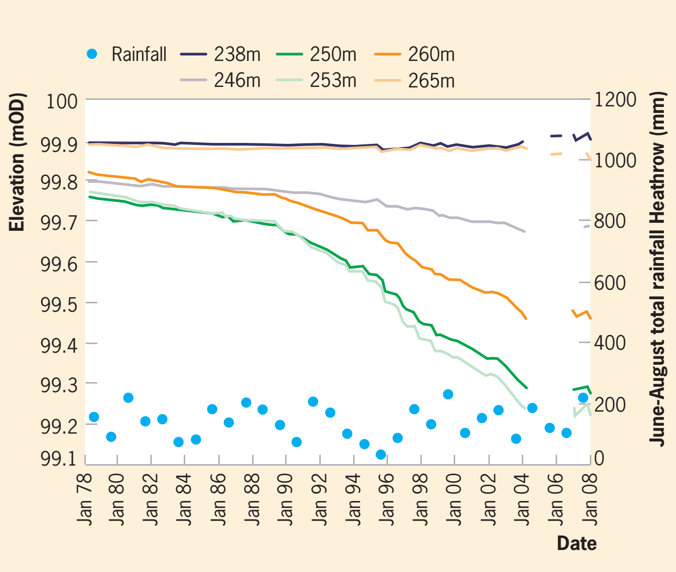

Figure 2: Wave wall settlement at unstable section

Soil nailing is suited to firm to stiff clays, but particular care is required in high-plasticity over-consolidated clays such as London Clay to avoid a degradation of the nail system due to cycles of shrinkage and swelling (Phear et al, 2005).

This paper presents the results of pore pressure and displacement monitoring from 2000 to 2011, together with wave wall levels for the period 1978 to 2008, for a particularly unstable and curious section of the downstream slope of Aldenham embankment dam, Hertfordshire, UK. The design and installation of a nailing scheme in 2003 is also described.

Site description and history

Aldenham embankment dam is located within Aldenham Country Park in Hertfordshire, UK (OS grid reference TQ167958) and is orientated approximately east to west with the reservoir on the south side (Figure 1). It was first completed in 1795 to supply compensation water after construction of the Grand Union Canal (Faulkner, 1972) and in 1802 its height and length were increased to today's values of 8m and 400m respectively and the reservoir capacity to 78,000m^3.

The contours of the natural landscape suggest that the original course of the river passed under the embankment at about 150m from the eastern shoulder, and there is also a kink in the embankment crest at this location which probably »

Figure 3: In January 1997 a 36m long sheet pile wall was installed in the upper upstream slope

« resulted from a misaligned meeting of both sides of the embankment over the original river course during construction.

The embankment fill was sourced from the weathered London Clay of the basin and slips occurred during construction, necessitating re-profiling to a shallower 1:4 slope, and have continued intermittently to the present day, more usually in the upstream slope.

The downstream slope around the location of the kink has become particularly unstable in the past 50 years. From about 1962, settlement of the wave wall at this point was reported. Wave wall levelling was undertaken in this area on a regular basis from 1978 onwards and its results are shown in Figure 2. Prior to 1988, average annual settlements between chainage 246m and 260m (the chainage is measured from the west shoulder) were about 5mm, but there were marked increases in the rate of settlement in 1989-90 (to 22mm/year) and 1995 (to 48mm/year).

These increases coincided with unusually dry summers, as indicated by the rainfall totals for the 23km distant Heathrow weather station, and settlements appeared to slow during comparatively wet summers (eg 1999, 2002), suggesting that the wave wall settlements were a result of suction-induced shrinkage in the clay fill. Interestingly, Smethurst et al (2012) noted that summer suctions in a London Clay cutting slope in Newbury were heavily dependent on summer rainfall amounts.

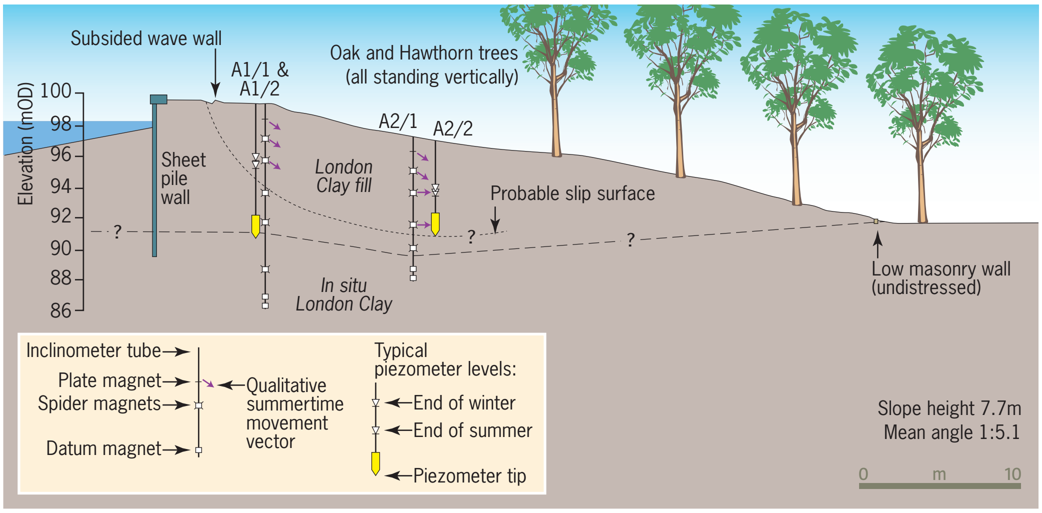

Figure 4: Cross-section of unstable downstream slope

In January 1997 a 36m long sheet pile wall was installed in the upper upstream slope around the unstable section to address the risk of loss of freeboard here due to the wave wall settlements, as shown in Figure 3. Earlier, in 1990, 2m deep gravel-filled drainage trenches at 5m centres were installed in the upper downstream slope of the whole dam to mitigate general deformations occurring in the crest footpath.

As shown in Figure 2, while installation of the sheet pile wall successfully restored the freeboard, neither this nor the installation of the gravel drains in 1990 arrested the wave wall settlements. Only the installation of soil nailing in the upper downstream slope of the unstable section in October 2003 finally halted the settlements.

An important factor governing pore water suctions in the embankment is the presence of trees on the lower downstream slope. Historical aerial photographs dating back to 1947 (Lees et al, 2013) show that trees were allowed to grow on all the downstream slope up to 1974 when, following recommendations under the Reservoir (Safety Provisions) Act 1930, the trees and shrubs were removed from the upper part of the slope (Kennard, 1975), probably due to concerns with tree roots penetrating the dam or erosion resulting from uprooting of trees in high winds (Hoskins and Rice, 1992).

Since 1974 to the present day, the upper slope has been grassed while only the lower slope has been covered by trees, as shown in the diagram of the unstable section in Figure 4. This resulted in a differential between upper and lower slope soil suctions that likely played a significant role in the initiation of a curious movement pattern at the unstable section.

Ground investigation

In order to design a stabilisation scheme at the unstable section (and to address smaller deformations in other sections of the embankment dam – refer to Lees et al (2013)), a sub-surface investigation and monitoring of the embankment dam was undertaken in February 2001. Five boreholes were sunk in the upper downstream slope at the unstable section generally to between 1m and 4m below dam foundation level. The London Clay fill encountered was generally a firm brown/grey sandy gravelly clay with decomposed roots and plant remains, similar to the London Clay fills encountered in old railway embankments that comprise clods of intact clay surrounded by a matrix of remoulded clay and foreign material (eg silt and sand) picked up during handling and transportation (O'Brien et al, 2004). The underlying London Clay was a firm grey/brown gravelly clay becoming stiff with depth.

Average liquid limit and plasticity index were 70% and 42% respectively with clay fraction recorded, using the sedimentation method, at 50%, which places the fill in the high plasticity category (BSI, 1999).

Gravimetric moisture content largely ranged between 30% to 40%, so at the lower end of the plasticity range and about half the liquid limit value which, according to Driscoll's (1983) crude estimate that desiccation of clay commences when the moisture content falls below about half the liquid limit value, suggests that at the time of the ground investigation – after a very wet winter – the clay fill of the upper downstream slope was at field capacity.

From routine isotropically consolidated undrained triaxial tests on nine samples of the London Clay fill clods, mean peak shear strength values of $\varphi' = 23°$ and $c' = 5$kPa were determined, which are similar to other values determined for London Clay fill (eg O'Brien et al, 2004). From direct shear testing, a residual shear strength of $\varphi' = 12°$ and $c' = 0$kPa was determined for the London Clay fill clods, with somewhat higher values obtained for clods containing foreign material.

Monitoring instrumentation and data

Vibrating wire piezometers connected to a data logger were installed in two boreholes at the depths shown in Figure 4. In two other boreholes, inclinometer tubes combined with extensometer spider magnets to measure vertical settlement were installed at the depths shown in Figure 4. Measurements from these were taken at various intervals as short as two weeks at the start to as much as 12 months near the end, depending on the availability of funds, using manual probes. Occasional gross errors in the data were corrected but otherwise the data appeared sufficiently accurate (estimated uncertainty ±2 mm), since it was possible to identify consistent trends in all the measurements.

Deformation monitoring results

Inclinometer A1/1 profiles for Summer 2003 with values reset to zero in April 2003 are shown in Figure 5. In spite of this being a particularly dry summer, it is clear that, remarkably, slippage occurred along a 5.5m deep plane. A 6m deep slip plane was also recorded in inclinometer A2/1 which, combined with the wave wall settlement, allowed the slip plane shown in Figure 4 to be drawn.

The inclinometers recorded similar movements in the summers of 2001 and 2002. This would account for the continual settlement of the wave wall and, since it is located well behind the slip, the ineffectiveness of the sheet pile wall at halting the settlement. However, there is no clear evidence of the toe of the slip emerging further down the slope and the trees on the lower slope all stand vertically.

The extensometer magnet elevation readings at 0.8m nominal depth have been combined with the horizontal deflection reading in the corresponding inclinometer A2/1 for the whole monitored period in Figure 6 to produce a vector plot of ground deformations at this point in the plane perpendicular to the dam. The overall approximate seasonal movements are indicated by the arrows.

There were cycles of downslope movement in each summer of 2001 to 2003, with by far the largest slippage occurring during the particularly dry summer of 2003, »

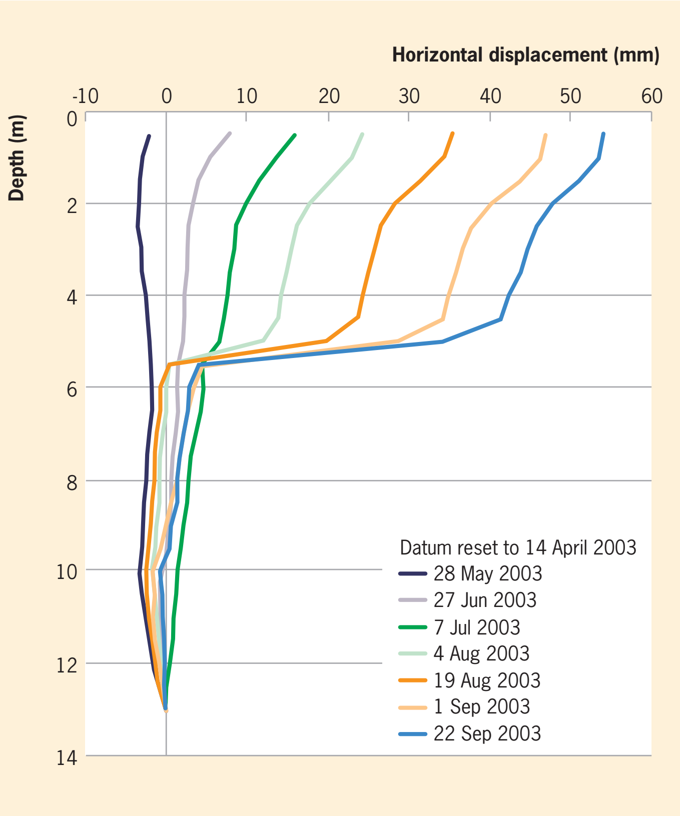

Figure 5: Inclinometer A1/1 – sample profiles from Summer 2003

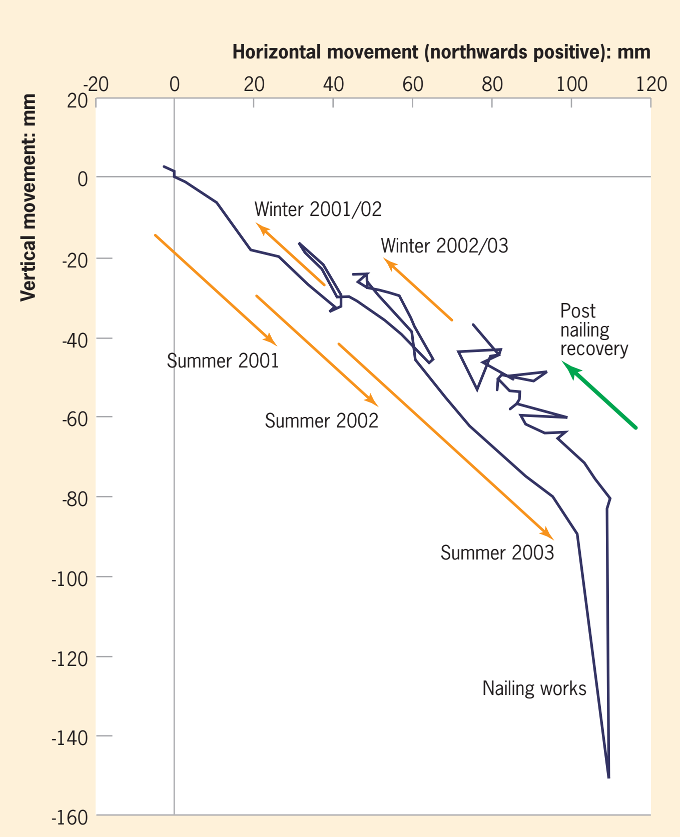

Figure 6: Measured deformations at 0.8 m depth in borehole A2/1 (April 2001 to March 2011)

« interspersed with partial recovery in approximately November to February each year. The A2/1 instrument suffered some disturbance, but no damage, during the nailing works and continued to provide data up to the last reading in March 2011. The plot shows that not only were downslope movements halted by the nailing but the low pre-stress imposed on the installed nails resulted in partial upslope recovery movements.

Pore water pressure monitoring results

Two piezometers were installed near to where potential slip planes were anticipated, as shown in Figure 4. At such depths, they were unlikely to record significant variations in pore water pressure due to vegetation-induced suctions near the surface. Nevertheless, since records show that the reservoir level varied by no more than 30mm above or 15mm below the overflow weir level of 98.18 mOD, it can be assumed that variation in pore pressure would have been caused by climatic changes.

The measurements from both piezometers are shown in Figure 7. A2/2 took longer to settle down, but both eventually recorded a small but regular approximate 5kPa seasonal variation reaching minima in late summer and maxima in spring. The lowest pore water pressure was recorded in the dry summer of 2003. A2/2 was damaged beyond repair during the nailing works but A1/2 continued to function and recorded about a 25kPa excess pore pressure due to the nail pre-stress that then dissipated over the following two years.

Also plotted on Figure 7 is a temporal plot of horizontal displacement recorded by inclinometer A2/1 at 1m depth. It confirms the paradox that significant downslope movements in the summer (prior to the nailing works in September 2003) coincided with reduced pore water pressure.

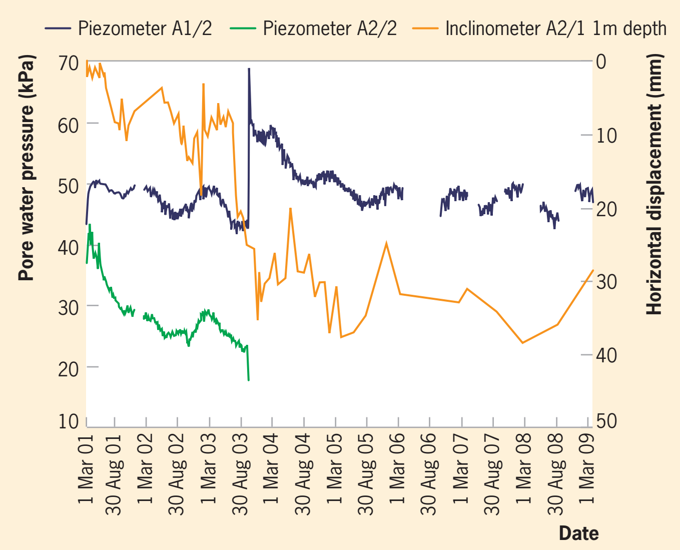

Figure 7: Piezometer data

Discussion of monitoring data

The phenomenon of significant downslope movement during periods of high soil suction, with the greatest movements being observed in the driest summers, is curious but has been observed before. Smethurst and Powrie (2007) recorded increasing bending moments in discrete piles used to stabilise an unvegetated Weald clay fill railway embankment during the dry summer months of 2001 and 2003. The authors suggested that suctions caused by mature trees at the toe of the slope caused the clay to crack, thereby removing support for the slope. Andrei (2000) also noted significant accumulations of track settlements during the summer months on a clay fill railway embankment wherever trees were located on its slopes.

It is likely also at Aldenham dam that clay shrinkage caused by trees on the lower slope removed support to the treeless upper slope, resulting in ratcheting downslope movements. But why did a deep-seated slip failure occur at this unstable section and not in other parts of the dam in recent years? Higher magnitude pore water pressure cycles were likely here due to the large number of high water demand English oak trees located on the lower slope at the unstable section. The resulting loss of toe support each summer either initiated progressive formation of a slip plane or caused reactivation of one of the many reported slip failures that occurred during construction when the slope was steeper.

The clay fill at the unstable section – on the probable former river course – may have been placed in difficult, wet conditions, leading to more slip failures during or soon after construction, prior to re-profiling. Positive pore water pressure along much of the slip plane even during the summer months, due to seepage from the reservoir, also contributed to the movement mechanism.

Soil nailing scheme

Clearly, stabilisation of the unstable section of the downstream slope was required in order to arrest annual downslope movements of at least 20-40 mm. This would negate the risk of eventual uncontrolled release of water from the reservoir if the movements continued and the slip grew beyond the extent of the sheet pile wall, as well as saving continual maintenance to restore the crest footpath. However, the embankment dam forms an important part of the popular Aldenham Country Park and it was important that the stabilisation scheme preserved the park environment. This ruled out removal of the trees (which also had preservation orders on them) and heavy structures, such as an embedded wall in the downstream slope, that may have been ineffective in any case.

The preferred option was soil nailing because the nails could be spatially distributed to support the entire slip mass to prevent crest settlement and because the nails could be buried and grassed over to restore the park environment.

There were concerns regarding deterioration of the nailing scheme over time due to the repeated shrinkage and swelling of the clay fill, as described subsequently by Phear et al (2005). To address these concerns, the following measures were taken:

- nail centres were quite small at 1.5m on a triangular grid;
- quite long nails (12m) were used to extend well beyond the active zone in the fill and into the intact London Clay a 0.35m thick granular layer stabilised by a single biaxial geogrid (Tensar SS40) at the base below the nail plates was placed over the nail heads to help distribute

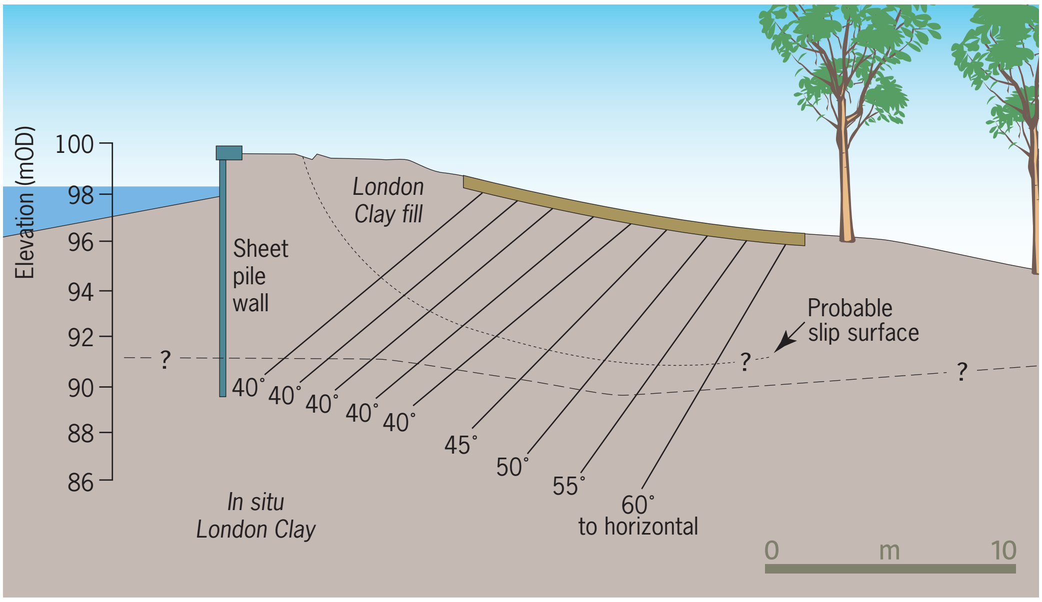

Figure 8: Nailing scheme

the nail loads evenly over the surface of the slope;

- effective stress, residual shear strength parameters ($\phi' = 12^\circ$, $c' = 0$ for the upper and lower fill; $\phi' = 14^\circ$, $c' = 0$ for the remainder containing a higher proportion of foreign material) were adopted for the clay fill in the slip circle analyses and bond strength design, and constant volume strength ($\phi' = 22^\circ$, $c' = 0$) for the intact London Clay;
- monitoring of slope movements would continue for at least five years after completion of works, and wave wall level monitoring in perpetuity.

The nailing scheme was designed using Talren 97 version 2.2 (Bishop method slip circle analysis) and in accordance with Clouterre (1991). Hydrostatic conditions were assumed below a groundwater level that was based on the reservoir level, the average piezometer readings and assuming groundwater level at ground surface downstream of the dam. Zero pore pressure was assumed above groundwater level.

When analysed as a whole, the downstream slope had adequate stability, even with residual shear strength. However, when shrinkage of the toe was included, a slip circle with 0.98 factor of safety whose geometry matched the profile determined from the inclinometers was obtained only by constraining slip circles to pass at depth to avoid shallow failures.

Toe shrinkage was modelled by replacing the tree-covered part of the slope for up to 3m depth by a surcharge of equivalent self-weight, thereby removing the lateral support to the upper slope but maintaining the weight at the toe. This was equivalent to forming a 3m deep tension crack immediately upslope of the trees and its success at back-analysing the slip failure lends weight to the argument that shrinkage of the tree-covered toe each summer was the trigger mechanism for the slope movements.

The nailing arrangement was designed in Talren using the same method of modelling the toe shrinkage, with a target overall factor of safety of 1.5. The shear resistance of the nails was ignored. The nailing arrangement shown in Figure 8 was found to be the most efficient (factor of safety 1.57, maximum nail load 65kN), although the upper nails were constrained by the sheet pile wall and could not be installed at their most efficient angles.

The nails comprised 32mm diameter hollow manganese steel bars, installed with sacrificial drill bits on their ends and with a water-cement flush fed through the hollow bar (to cause less swelling and softening of the borehole walls compared with water flush).

Once each nail had been installed, a Portland cement grout was injected down the hollow bar until all drill flush and debris had exited the borehole. Three test nails were installed on site with 4m long bond lengths and achieved satisfactory failure loads (two at 130kN, one at 160kN). Thirteen (or 5.3%) of the 243 working nails were test loaded to 150% working load and all passed with no significant deflection. All the working nails were tensioned to about 10kN when locked off.

As shown in the displacement measurements at borehole A1/1 in Figure 6 as well as the wave wall settlement data in Figure 2, the soil nailing scheme was successful in halting the accumulation of downslope movements. This was also borne out by deflection measurements at other depths and in another borehole (refer to Lees et al (2013)).

Indeed, the pre-stress in the installed nails resulted in an upslope recovery (Figure 6) of 35mm in the first year and a further 20mm in the following six years. No net annual downslope movement was recorded by any of the monitoring instruments in the 7.5 year monitoring period following nail installation.

Conclusions

A curious slope movement pattern has been recorded at one unstable section of the downstream slope of an old clay fill embankment dam. A slip plane in the upper slope was reactivated each summer and the drier the summer, the greater the downslope displacement. This was caused by shrinkage of the tree-covered toe of the embankment due to soil moisture deficit in the summer, leading to loss of support to and reactivation of the grassed upper slope slip mass. It could not be determined whether the slip plane developed progressively from the pore suction cycles or whether it was pre-existing from the reported post-construction failures.

The toe of the embankment is stable due to reinforcing effect of the tree roots and probable persistent soil moisture deficit, so the toe of the slip plane did not reach the ground surface, resulting in a bulging of the mid-slope profile evident from topographical surveys.

An unobtrusive slope stabilisation scheme was required on this country park site, so a soil nailing scheme was designed and installed to support the upper slope slip mass, and then grassed over. Inclinometer and wave wall settlement data in the seven years following nail installation have shown the scheme to be successful, even resulting in some upslope recovery movements.

Acknowledgements

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