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Improving the Climate Resilience of Railway Earthworks: Case Studies from Southeast England

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Abstract

Transportation earthworks infrastructure is adversely affected by global warming induced climate change. Seasonal shrink/ swell deformation, exacerbated by increasingly frequent and exaggerated wetting and drying cycles are resulting in failures of railway earthworks. Their reliability is already degraded by age and limited by the original construction methods and materials. This paper provides a review of the measures employed to improve the resilience, safety and reliability of railways earthworks in southeast England during the Network Rail Control Period 6 (CP6). The paper presents examples from three categories: (1) mitigation measures following an earthwork failure, which has limited indication of instability prior to failure; (2) mitigation measures taken pre-emptively on earthworks which are identified as being at risk of failure due to progressive movements indicated by monitoring data; (3) mitigation measures on earthworks that are considered at increased risk of likely failure accelerated by climate change. The project examples consist of both cuttings and embankment in overconsolidated clays and weathered weak rocks situated in challenging topographies. The paper highlights the collaborative effort between the principal framework contractor (BAM Nuttall), the designer (Tony Gee and Partners) and the asset owner (Network Rail) to maintain a railway in an increasingly demanding climatic environment.

Keywords: Slope remediation, earthworks cutting and embankment, soil nailing

1. Introduction

The majority of the Network Rail's (NR) cutting and embankment assets in the southeast of England are now more than 150 years old. Originally built in the Victorian era, they have been experiencing frequent stability issues of both a shallow and deep-seated nature. Increasingly frequent and extreme weather cycles, linked to climate change, are believed to be contributing to this deterioration.

As part of UPCP18 projections, Met Office (2022) reported that winters in the UK, for the most recent decade (2009-2018), have been on average 5% wetter than 1981-2010 and 12% wetter than 1961-1990. The report stated that total rainfall from extremely wet days does not show significant change for most of southern and eastern England but Scotland. Kendon *et al.* (2021) agreed with the findings of the UKCP18 but emphasised that heavy rainfall is a complex variable to monitor and statistically correlate due to its potential to be highly localised. They pointed out that more heavy rain events had been recorded in the most recent decade than in earlier decades. For instance, 2020, particularly in February, had widespread heavy rainfall events across the UK. Met Office (2022) noted that the most recent decade (2009-2018) was approximately 1 °C warmer than the pre-industrial period (1850-1900).

Birch and Evans (2018) observed, from their review of the period 2003 to 2014, that there is a strong correlation between NR earthworks failures and winter rainfall. Similarly, NR in their 'Review of Earthwork Management' report (NR, 2021), made a similar observation of increased asset failure frequency and elevated/ prolonged rainfall. The NR report provides the analysis for the winter of 2019/2020 which is identified as one of the wettest on record. During September to December 2019 in southeast England, the rainfall as a percentage of long-term averages since 1862 was more than 130%, with a peak of 154% in December alone. However, comparing with the averages of asset instability for this 17-year period, three times more cuttings and two times more embankments failed in the autumn and winter of 2019/2020. Failures have also been observed in summer particularly after a dry spell followed by more intense rainfall, as is being experienced presently following the record-breaking dry summer of 2022.

In cuttings, failures, particularly shallow slips, are thought to be primarily due to coupled precipitation related softening and age-related weathering, with contributing factors being originally steep cutting slope angles and absence of drainage (or, if present, blocked drainage). Seasonal changes of soil moisture content, exacerbated by unusually heavy or prolonged precipitation periods, causes pore-water pressure increase within the slope and plastic shearing leading to dilatant softening with loss of suction. As a result, first-time slips and/or reactivation of older slips occur (Crabb and Atkinson 1991). Based on the earthworks asset monitoring by NR, the Wealden geology in Southeast England, characterised by high plasticity over-consolidated clays and weak rocks, is reported as being vulnerable to earthwork failures (NR, 2021).

NR (2021) reported that embankment failures occur in both predominantly fine-grained ('cohesive') fill (with more deep-seated failures due to softening/development of residual shear planes following prolonged rainfall in winter) and coarse-grained ('granular') fill (spreading and erosion). The main reason for embankment failures is associated with the original construction method using material sourced from adjacent cuttings placed by side-tipping (or end-tipping) at steep angles without compaction. They were also commonly built on weak basal founding levels without preparing the ground to provide a competent founding formation. The embankments were often built with such steep slopes which typically rely on suction generated within lumps of excavated clay. This resulted in after-construction consolidation and shear failures once suction disappeared, which were then mitigated by re-grading the embankment via tipping ash and placing more ballast over it (Taylor *et al.*, 2015; Briggs *et al.*, 2017; Birch and Evans, 2018; Standing *et al.*, 2021). The embankments built from high plasticity clays are particularly affected by seasonal water content change related shrink and swell, and associated strain-softening. Stirling *et al.* (2021) observed that cyclic wetting and drying causes microstructural changes to soil fabric resulting in loss of suction and therefore strength. These microstructural changes lead to macrostructural features, such as cracking. Skempton (1985) and Lupini *et al.* (1981) previously stated that cracking allows the rainwater ingress to embankment slopes and causes local stress concentration and softening that can result in local movements and progressive failure. Stirling et al. (2021) pointed out that the rate of such deterioration is nonlinear, with the greatest observed change occurring after the initial, primary drying of newly compacted cohesive fill. However, with extreme weather events further changes will continue to develop with continuous wetting and drying cycles.

This paper presents four case studies in southeast England where mitigation measures were designed by Tony Gee and Partners ('TGP') and constructed by BAM Nuttall ('BAM') during the NR Control Period 6 SMD Framework in order to increase the resilience of earthworks assets to climate change and to improve their design life. The case studies are grouped under three categories: (1) emergency response following an earthwork failure through the examples of the High Brooms Cutting and the Edenbridge Embankment and; (2) mitigation measures taken pre-emptively on earthworks which are identified as being at risk of failure due to progressive movements indicated by continuous monitoring with an example of the Balcombe Embankment; and (3) mitigation measures on earthworks that are considered at increased risk of failure accelerated by climate change with an example of the Bracewell Road Embankment. The locations of the earthworks are shown in Figure 1.

2. Emergency response following slope failure

2.1 High Brooms Cutting

High Brooms cutting is located on the Tonbridge to Hastings line, which was opened in early 1850s, at the north of High Brooms and the south of Tonbridge, Kent. Original cutting was built into a geology consisting of the Wadhurst Clay Formation ('Wadhurst Clay') overlying the Ashdown Formation, leaving a very narrow cess. The cutting slope angles here were 38° to 45°. The cutting has experienced previous first-time slips in the past. In 2014 autumn/winter, the Wadhurst Clay section of the slope which is overlying the Ashdown Formation approximately at track level slipped. Emergency repair comprised re-grading and soil nailing the Wadhurst Clay slope with installation of a crest drain, albeit set approximately 8 m back from the crest edge, along the boundary fence extending beyond the failed section to the north, and a flume.

In February 2020, the slope failed at three locations following intense rainfall during 2019/2020 November to January, as shown in Figure 2. The main slip was approximately 15 m away from the end of 2014 repair works. The failures were translational first-time shallow slips.

Figure 1: Case study locations

The main slip extended to the mid-upper slope, but not as far up as the crest. It was approximately 8 m wide and less than 1 m deep. The slope height at the failed sections was approximately 12 m (including 2.7 m high sloping ground to the crest at 11.5° and 9.4 m high slope at 41°). The other two slips were narrower, but otherwise of similar dimension. NR immediately responded to protect the railway by placing ballast bags at the toe of the slope and installing monitoring survey points along the failed sections. The track speed limits were also lowered.

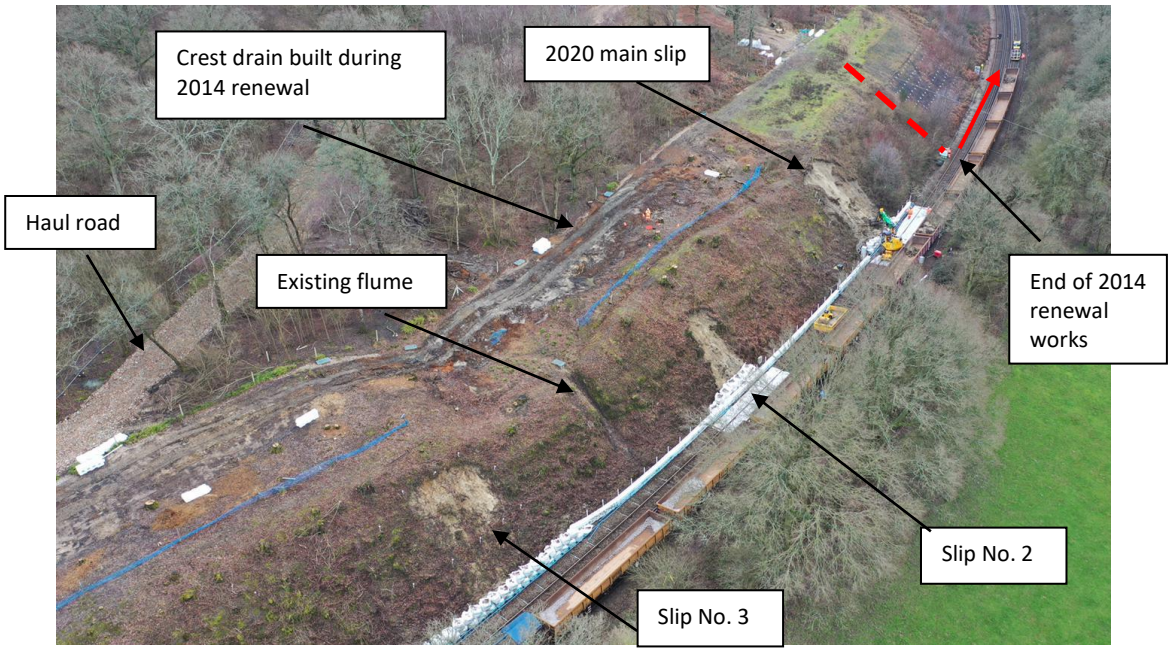


Figure 2: High Brooms cutting after February 2020 slips

Following the site visit by Tony Gee and a NR geotechnical asset engineer, it was decided to stabilise the Wadhurst Clay slope via regrading and soil nailing. Two rotary drilled boreholes encountered firm to stiff friable high plasticity clay overlying highly to completely weathered mudstone interbedded with siltstone and

occasionally ironstone. The base of the Wadhurst Clay was thinly laminated to thinly bedded siltstone. The Ashdown Formation was moderately weathered, extremely weak to moderately weak sandstone with interbedded thin siltstone and mudstone.

2.1.1 Mitigation solution

Back-analysis was carried out using Wadhurst Clay's critical state strength by capping the 'frictional' component of the strength at the critical state angle and introducing a component of mobilised effective 'true' cohesion (Take and Bolton, 2011). Angle of shearing resistance ϕ' was taken as the critical state value in the back-analysis and the c' was adjusted for a safety factor ≤ 1.0. Clearly, there should not be a cohesion in volumetric softening (i.e. on the dry side of the critical state line). However, the difference between peak and critical state strength of the clay slope is simplified for design purpose by introducing cohesion. Because of lack of pore water pressure monitoring within the slope, the slope is considered partially saturated with pore water pressure coefficient, r_u=0.2 for simplicity. The back-analysis demonstrated that approximately 1 m thick shallow slip within the Wadhurst Clay was mobilised similar to the actual slip at ϕ'=25° and c'=4.5 kPa. Below this zone, c'=7 kPa was used.

Soil nails and the face plates were designed according to BSI (2017) and Phear *et al.* (2005). The top 1 m slice of the clay slope was considered to be in a "long-term" softened state, therefore it was modelled with the parameters given above. The slope was proposed to be graded to 31° prior to installation of soils nails. 9.5 m long Dywidag R32-280 nails in 100 mm diameter bores were proposed to be installed in a staggered diamond pattern with a minimum spacings of 1.2 m vertical and 2.0 m horizontal at 25° inclination. Structural flexible facing comprised 2 mm diameter steel wire mesh connected to the nail heads with a 15 mm thick 300 mm square galvanised S275 grade steel plate.

2.1.2 Further slip in 2021 and revised mitigation solution

Due to unexpected circumstances brought by the pandemic, the work progress was slower than planned. In January 2021, the previously stable upper slope of the main slip failed further. The translational slip was approximately 9 m wide and 0.5 m to 1 m deep at post-failure slope angle between 32 and 35°. NR took the possession of the line and mobilised BAM to deliver the approved mitigation scheme within the CP6 framework contract. During the re-grading of the slope, it was identified that the thickness of the Wadhurst Clay reduced to approximately 1 m towards north.

During a site meeting between the engineers from TGP and BAM and the NR geotechnical asset engineer, the rock exposure was inspected, and it was decided to terminate re-grading of the Ashdown Formation and instead to design a rock netting containment system. Re-grading works left the siltstone and the underlying sandstone exposed, resulting in locally very steep overhanging fractured rock. Figure 3 shows the boundary between two formations exposed during re-grading at the main slip location, and the spider excavator, with limited head room requirements, utilised to work under the high-voltage overhead electric power lines. Further discussions between the designer and the contractor optimised the netting solution considering the current stability of the Ashdown Formation slope. To mitigate likely future rock fall due to the more frequently occurring extreme weather events, a passive netting system was installed in the area of the Ashdown Formation slope that was not re-graded, and an active netting system was installed on the re-graded rock cutting.

2.2 Edenbridge Embankment

The rail embankment supporting the Redhill to Tonbridge Line between Godstone and Edenbridge suffered a catastrophic failure following an extended period of heavy rainfall during December 2019 which caused flooding at the embankment toe (Figure 4). The embankment was constructed in 1830s and is one of the earliest rail embankments in the UK. It is approximately 10.5 m high with average slope angles of 27°. The embankment is likely founded over Alluvium overlying the Weald Clay Formation ('Weald Clay'). The embankment was constructed from reworked Weald Clay from nearby railway cuttings. Ashy ballast was placed on the cohesive fill over time to compensate for ongoing settlement. The CPTs undertaken indicated approximately 2 m thick alluvial sand is located directly under the fill and overlying the Weald Clay.

The current course of the River Eden crosses beneath the embankment via a culvert at the east boundary of the slip as shown in Figure 5. The river is the source of high flood probability at this location (yellow shaded area in Figure 5). In addition, the runoff collected into an overcapacity ditch passes under the railways via a NR culvert

then runs parallel with the embankment in an open ditch (red dashed line in Figure 5). The ditch originally discharged into the River Eden, but over time it eroded creating an open channel to the river that has been flooding the embankment toe at the Upside (south) as well.

Figure 3: Regrading works at High Brooms Cutting

There is also the poorly functioning culvert from the Downside to Upside (orange dashed line) that discharged into the river. It is probable that flooding related porewater pressure changes and softening at and above the embankment toe contributed to the failure of the embankment. Given the age of the embankment, it is likely that the mechanism behind the failure was due to non-recoverable plastic strains developing between dry and wet seasons in the slope coupled with strain-softening due to seasonal pore water pressure changes (Kovacevic et al., 2001; Take and Bolton, 2011).

Figure 4: Failed Edenbridge embankment

Based on the experiments summarised in Kovacevic et al. (2001), plastic strain can propagate towards the embankment core, eventually leading to progressive deep-seated failure starting from the embankment toe which passed its critical strength state and is at or close to its residual strength.

2.2.1 Mitigation solution

The remediation work included the rebuilding of the Downside of the embankment. BAM carried out the works 24/7. Due to the Downside (north) of the embankment being inaccessible, a section was cut into the existing embankment to allow construction plants to be mobilised at the failed slope. The failed embankment fill on the Down side was excavated and removed, and MCHW Class 1A granular fill was placed to rebuild to the slope. The slope was excavated and benched into the existing embankment core at a maximum average slope angle of 45°. The finished slope angle was 25° to ensure a similar slope profile to the original embankment. The base of the embankment reconstruction was formed by a 600 thick drainage layer tied to the refurbished toe drain

draining to a NR culvert. The lower 2 m of the slope was protected by a rock mattress to reduce the impact of future flooding on the embankment. Figure 6 shows the site works.

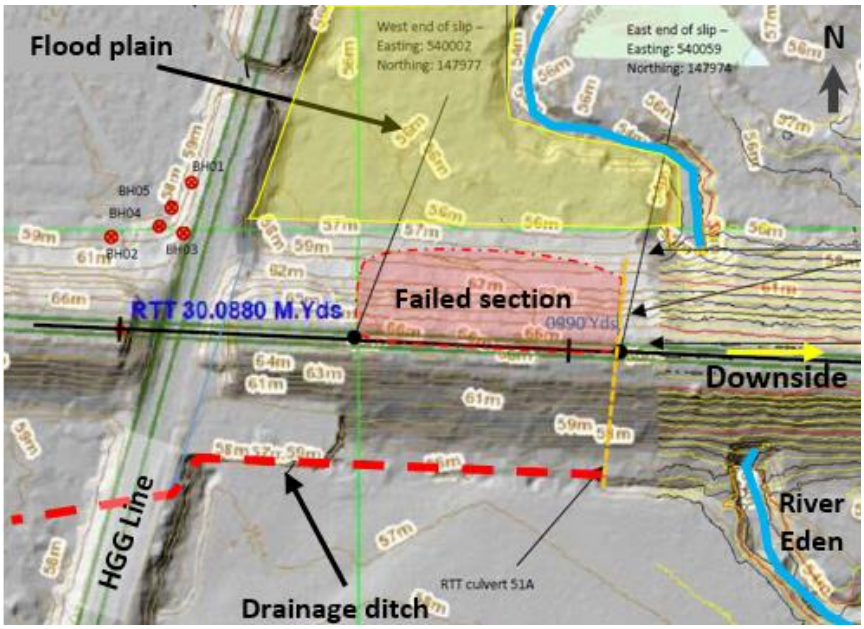


Figure 5: Edenbridge embankment layout

The back-analysis indicated that despite the reduced embankment height at the Upside due to natural fall of the ground, there was also a reduced safety factor. To increase the resilience of the whole embankment, excavated original clay fill which was classified as MCHW Class 2A material was used to build a new slope at 14°. The new fill was also founded on drainage layer similar to that installed on the Downside. A new drainage system was design and installed on the Upside to provide further resilience by preventing the Upside toe flooding from the HGG line culvert stream.

3. Pre-emptive mitigation measures at high-risk earthworks

3.1 Balcombe Embankment

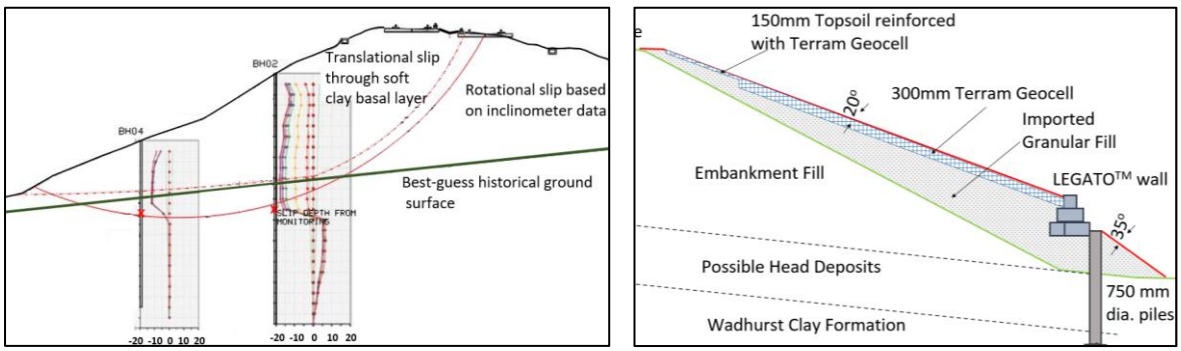
The site is located in West Sussex, approximately 500 m south of Balcombe Station on the Victoria to Brighton line. The embankment was built over a 'sidelong ground'. It is maximum 15 m high, reducing to 8.5 m. The slope angles are typically between 34° to 37°. In 1975, a deep-seated rotational slope movement was observed on the Upside slope where the embankment height was greatest, and remediated using grouting. The embankment has been showing movements resulting in track quality deterioration and was subjected to survey point monitoring since winter 2016/2017.

Further deterioration was observed and in 2020 six inclinometers spread across approximately 200 m were installed to determine underlying cause. Significant movements recorded in January-February 2021 (5 mm) and June-July 2021 (6 mm) correlated to rainfall during these periods. The time at which the reduction in track quality occurred supported the hypothesis that the accelerated movements were related to significant rainfall events. But there was otherwise continuous downslope movement in the order of 0.5 mm to 1.5 mm per month. Slope movement of the embankment possibly resulted in shear surfaces becoming increasingly slickened by time. The rate of movement could have accelerated and, under certain conditions, such as heavy and prolonged rainfall event, the embankment could have failed rapidly.

The embankment spanned over a small valley of Head deposits overlying the Wadhurst Clay. The fill consists of sandy silty clay topped with a variable thickness of track ballast over time. The ground investigation indicated the presence of variable embankment fill overlying Wadhurst Clay, with no reference to Head deposits. Considering the shapes of the mechanisms observed via inclinometer readings, it is probable that a layer of softened material existed at the interface between the embankment fill and the founding formation. Further investigation using CPTs indicated the presence of soft, possibly Head at the interface.

Figure 6: Edenbridge embankment during the works

Figure 7(a) shows slip surfaces interpreted from the inclinometer data recorded from 2016 to 2021. Reviewing all the inclinometer data, a very similar rotational movement to that considered in the mid-1970s, and treated with grouting, appeared to have reactivated. Additional translational slip mechanism was also observed based on two inclinometers, which were likely occurring at the foundation level.



(a) (b)

Figure 7: (a) Slip surfaces and inclinometer monitoring data at Balcombe embankment (b) Proposed design

It was considered likely that surface water runoff flowing down the sidelong ground which did not enter the culverted stream might have interacted with the failure surfaces by either percolating through the embankment from the slope side or moving immediately under the embankment along the original sloping ground interface. Rainwater was also locally ponded within the embankment as indicated by shallow water strikes in the boreholes. Such pockets of water might have also contributed to the sustained slope movement.

3.1.1 Mitigation solution

The geotechnical parameters used in the design were derived from the ground investigation data and the back-analysis. This analysis simulated deep-seated translational and rotational failure mechanisms similar to the inclinometer monitoring data. Together with the CPT results and ring shear tests carried out on the founding stratum, for the design of the mitigation solution embankment fill effective strength was determined as c'=4 kPa and ϕ'=19°. The strength of the soft interface between the fill and the underlying Head deposits were taken as c'=2 kPa and ϕ'=17°. Piezometric level was taken at the natural ground level. Pore water pressures within the embankment was simulated using r_u=0.2. In order to arrest the embankment movement and prevent eventual failure, a piled retaining wall consisting of 750 mm diameter cast in-situ reinforced concrete piles with 1250 c/c spacing were proposed, as indicatively shown in Figure 7(b) for 13 m high section. The pile lengths were 13 m to 16 m long toed into the Wadhurst Clay mudstone. The existing slope was cleared and benched to allow placement of 'granular' fill to form a shallower 20° slope in order to prevent future shallow slips due to heavy and prolonged precipitation.

The fill was retained by 3.2 m high gravity wall constructed using Legato™ concrete blocks behind the piled wall. During the pile installation, the concrete blocks were also used to form the temporary piling platform.The piled retaining wall was buried completely as part of the granular fill placement to the existing embankment toe (Figure 8). A drainage channel was also built at the toe of the embankment to drain the water and prevent further erosion.

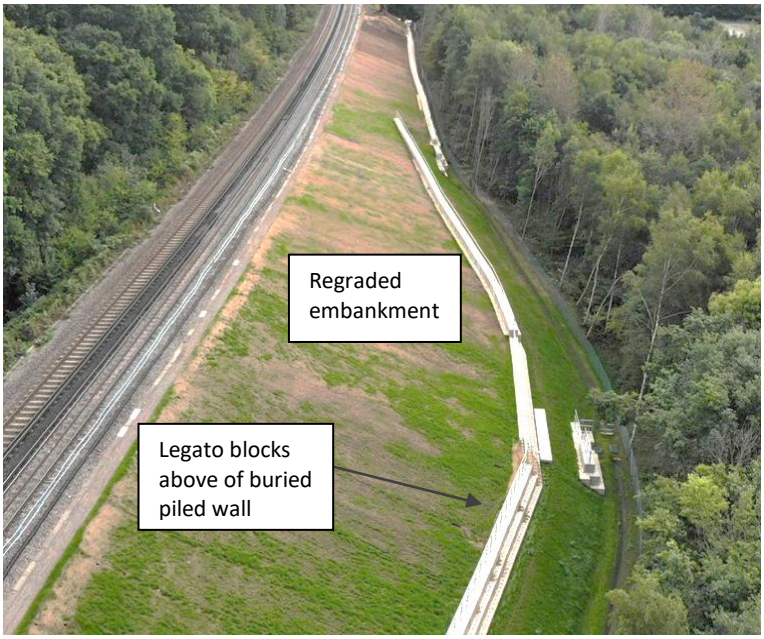


Figure 8: Balcombe embankment after completion

4. Mitigation measures on earthworks at increased risk due to climate change

4.1 Bracewell Road Embankment

The Bracewell Road embankment is located on the West London Line which was opened in 1844. The embankment is located between the Shepherd's Bush Overground Station and the North Pole Junction. It is approximately 230 m long and historically experienced poor ride quality. The embankment consisted of old ashy ('dirty') ballast with variable thickness overlying the embankment fill of London Clay origin. Therefore, it is highly likely that the fill consisted of large over-consolidated clay lumps which were either uncompacted or compacted poorly, thus the stability of the embankment after construction depended on suction. The founding stratum is the London Clay Formation bedrock. The slope in question is on the Upside (east) of the embankment and is approximately 5 m high with slope angles of up 40° on the upper slope (old ballast), slackening to about 20° to 30° degrees on the lower slope.

NR records indicated that the embankment experienced local slip failures and was reinforced in 1999-2000 by installing soil nails on the Upside upper part of the slope with steel plate facing and covered with facing mesh and topsoil. The Downside embankment slope was also likely subject to previous remedial works of unknown type. Sections of the embankment also included a toe drainage. During a site visit, indications of local shallow creep movements were observed. There were slope movement indicators such as displaced troughing, tilted OLE and leaning trees. However, there was no back scar at the slope crest observed, nor toe bulging. The likely future failure mechanism of the embankment is similar to one discussed in Section 2.2 for Edenbridge embankment. The difference here is that the London Clay fill has higher plasticity and lower critical state strength. The strength cycles resulting in plastic strains are associated with shrinkage of the high plasticity clay in dry season creating inward-downward movements followed by swelling resulting in outward-upward movement. Climate change may speed up and amplify the impact of these cycles in the embankment.

Peg monitoring was undertaken at the crest. There was a clear acceleration in movement following Storm Alex in October 2020 (see red arrow in the figure) indicating the stability of the embankment slope was vulnerable to heavy rainfall events. Peg monitoring also showed that at the soil nailed section of the slope the

displacements were occurring but albeit at a significantly reduced rate compared to the adjacent unreinforced slope. Additionally, several boundary transgressions over the last several years by adjacent property owners, which consisted of excavations up to 3 m at the NR boundary line, may have also contributed to the embankment deterioration. Although not detailed here, it is possible that the existing vegetation provided some slope stability.

4.1.1 Mitigation solution

Ground investigation included boreholes with inclinometers and vibrating wire piezometers, CPTs and GPR survey at track level. GPR survey revealed possible subsidence within the embankment with a series of short irregular poorly defined secondary interfaces consistent with an unstable track bed. One inclinometer showed lateral movement of up to 8 mm approximately at 2.5 m depth below track level. A back-analysis was carried out using geotechnical parameters from the ground investigation and calibrating them to develop a deep-seated slip within the embankment fill. Options assessment was carried out to demonstrate the resilience of the slope stabilised with (a) soil nails which were already applied to the part of the slope, (b) cantilever sheet pile wall and (c) spaced piles. Considering the site constraints and impact of the works on the working line, and giving particular emphasis to likely future boundary transgressions by third parties, embedded retaining wall solution comprising spaced-piles was considered to offer long-term resilience, and also being simpler and more economic to build within the site constraints.

Cast-in-situ 450 mm diameter reinforced concrete CFA piles were constructed approximately 4.5 m from the slope crest at minimum centre to centre spacing of 1125 mm to maximize arching. The piles were 8 m long and embedded sufficiently below the calculated slip surface within the London Clay Formation. The piles were placed at approximately the mid-height of the embankment to ensure a “long pile” beneath the failure surface. Some pile locations had to be re-adjusted on site to avoid unforeseen obstructions. The section of the upper slope without existing soil nails was found not to be compliant with the standards, therefore soil nails were proposed to stabilise the upper slope during the next phase of works together with a Grundomat cess support wall to be anchored back into the embankment fill which will be constructed at a future date.

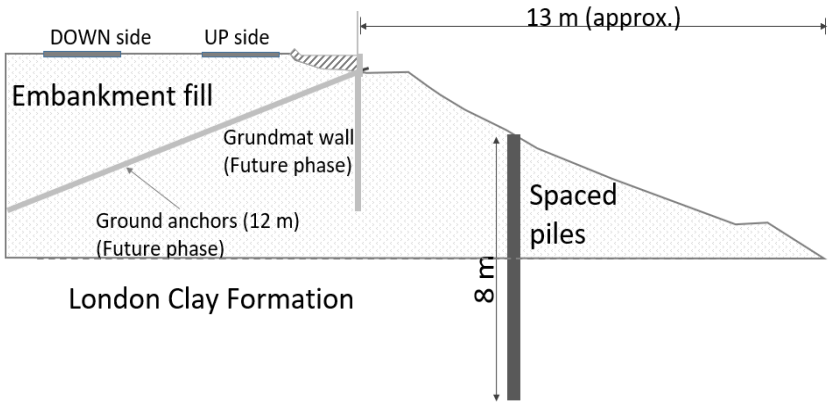


Figure 8: Bracewell embankment proposed design

5. Conclusion

The paper summarised examples of NR earthworks that are over 150 years old and either failed or at high risk of failure which was further increased as a result of climate change related extreme rainfall. The mitigation measures were developed considering various options to identify a constructable design. This required assessment of the causes of the failure via modelling, including numerical modelling, which utilised ground investigation and monitoring data. The mitigation option often rely on pragmatic and innovative solutions which were also necessitated by site and programme constraints and cost considerations. The preferred option in each case could only be achieved with close collaboration between the designer, the contractor and NR.

6. Acknowledgements

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7. References

Birch GP and Evans ER (2018). Railway earthworks with particular emphasis on the behaviour of clay embankments in the South east. *Ground Engineering* March 2018, 24-31.

Briggs KM *et al.* (2017) Failures in transport infrastructure embankments. *Engineering Geology* 219: 107-117, 10.1016/j.enggeo.2016.07.016.

BSI (2017) BS 8006-2:2011+A1:2017. Code of practice for strengthened reinforced soils. Part 2: Soil nail design. BSI, London, UK.

Crabb GI and Atkinson JH (1991) Determination of soil strength parameters for the analysis of highway slope failures. In *Slope Stability Engineering: Developments and Applications* (Chandler RJ (ed.)). Thomas Telford, London, UK, pp. 13–18.

Kendon M *et al*. (2021). State of the UK climate 2020. *International Journal of Climatology*, 41(Suppl. 2): 1–76, 10.1002/joc.7285. (open access).

Lupini JF *et al.* (1981). The drained residual strength of cohesive soils. *Géotechnique* 31(2): 181–213, 10.1680/geot.1981.31.2.181.

Met Office (2022). *UKCP18 UK Climate Projections: Headline Findings*, revision V4.0, Exeter, UK. See https://www.metoffice.gov.uk/binaries/content/assets/metofficegovuk/pdf/research/ukcp/ukcp18_headline_findings_v4_aug22.pdf for further details [last accessed 14/02/2024].

NR (2012) *A Review of Earthworks Management*. Network Rail, February.

Phear A *et al.* (2005). *Soil nailing - best practice guidance*. Ciria C637 Report, Ciria, London, UK.

Skempton AW (1985). Residual strength of clays in landslides, folded strata and the laboratory. *Géotechnique* 35(1): 3–18, 10.1680/geot.1985.35.1.3.

Standing JR *et al.* (2021) Observed behaviour of old railway embankments formed of ash and dumped clay fill. *Géotechnique* 71(11): 938-956, 10.1680/jgeot.19.SiP.045

Stirling RA *et al.* (2021). Weather-driven deterioration processes affecting the performance of embankment slopes. *Géotechnique* 71(11): 957-969, 10.1680/jgeot.19.SiP.038.

Take WA and Bolton MD (2011). Seasonal ratcheting and softening in clay slopes, leading to first-time failure. *Géotechnique* 61(9): 757–769, 10.1680/geot.9.P.125.

Taylor GR *et al.* (2015) The use of laboratory testing in the characterisation of embankment clay fill from the UK rail network. *Proc of XVI European Conf Soil Mech Geotech Eng*. ICE Publishing, London, UK, pp. 527-532.

Kovacevic N *et al.* (2001). Progressive failure in clay embankments due to seasonal climate changes. *Proc of 15th Int Conf Soil Mech Geotech Eng*, Istanbul, Turkey. AA Balkema Publishers, pp. 2127-2130.