Stabilisation of the Stanton Lees landslip using an embedded pile retaining wall

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Abstract. A landslip occurred near the village of Stanton Lees, Derbyshire, UK, in November 2000 following a prolonged period of heavy rainfall. The slip resulted in the gradual, progressive down-slope movement of a steep embankment that was supporting an existing gabion retaining wall and a minor road which was subsequently closed to vehicular traffic. This paper describes the site; the sequence of events leading up to the start of the landslide; an interpretation of the ground conditions and onsite monitoring data; and the engineering works carried out in order to stabilise the slip and return the road to serviceability. The landslip occurred due to the superficial deposits moving over the weathered bedrock after the groundwater levels had been elevated following a prolonged period of heavy rainfall. A stability analysis indicated that the embankment was at a state of limiting equilibrium and its factor of safety on slope instability was found to be particularly sensitive to fairly minor changes in the groundwater levels. The remedial works replaced the existing gabion wall with a new cantilevered, bored pile retaining wall that comprised two rows of piles (600-mm in diameter), which were staggered in plan arrangement.

Key words. landslip, pile, rainfall, retaining wall, stabilisation.

1. Introduction

1.1. Background

A landslip occurred near the village of Stanton Lees, Derbyshire, UK, in November 2000 following a prolonged period of heavy rainfall. The landslip resulted in the failure of an earth embankment and the displacement and settlement of an existing gabion wall that had retained the earthwork supporting a minor road which was subsequently closed to vehicular traffic. Derbyshire County Council Consulting Engineers engaged Scott Wilson in May 2002 to carry out a geotechnical investigation in order to determine the cause of the landslip and to design the remedial measures that were necessary to return the road to serviceability. This paper describes the site; the sequence of events leading up to the landslip; an interpretation of the ground conditions and onsite monitoring data, and the design and construction of the engineering works.

1.2. Site Description

The embankment, which was constructed between 1955 and 1962, is located in front of the Cherry Tree cottage and, in plan, is about 90 m in length (Fig. 1). The site is located at an elevation of 180 to 190 mAOD on the eastern side of Stanton Peak. The natural hillside is characterised by pronounced undulations that extend from the foot of the gritstone crags located behind Stanton Lees village to Hillcarr Farm at an elevation of about 120 mAOD. Natural springs discharge off the hillside to the relatively level ground to the north of the landslide location before percolating underground as groundwater flow. Anecdotal evidence indicated that the embankment and the field below had carried surface flow after periods of heavy rainfall, with the flow originating from the western side of the landslip.

The embankment straddles a gentle topographic depression that is roughly centred on a cesspit at the toe of the slope and trends in a north-westerly direction directly in front of the Cherry Tree cottage (Fig.1). To the west of this depression, a benched spur follows the curve of the roadway up into the valley below the chapel building (Fig. 1). The embankment slope was at its steepest (35 to 40 degrees) directly in front of the cottage, with the slope angle reducing to about 25 degrees on either side (Fig. 2). The slope had a cover of grass, with sparse vegetation where it was at its steepest. Several trees and bushes, which were located at the toe of the slope on its western side, had bent trunks indicating the effects of previous ground movements. The embankment had shown signs of bulging and cracking over some of its length, with a small slip scar to a shallow slip evident at the eastern side of the main landslip, where the slope had been at its steepest.

1.3. LANDSLIP

The landslip occurred on the down-slope side of the single-carriage road which is located between the chapel building and Cherry Tree cottage (Fig. 2). The road, which falls in elevation by about 4 m between the chapel and the cottage, had a longstanding problem of instability. The slip was reactivated in November 2000 and again in February 2002, which caused an outward destabilising movement of up to 1.0 m of the existing gabion wall, Fig. 3(a) and (b). The southern part of the roadway had settled locally by up to 1.5 m along an affected length of about 60 m as a result. A tree located near the embankment crest to the west of the cottage had toppled over and the fill material that had been tipped into the sunken road, as a temporary repair, had settled noticeably.

The gabion wall (45 m in length and 2.5 m in height) had been constructed in stages. The oldest section, located nearest to the chapel building, having been built between 1985 and 1986. The most recent section, located directly in front of the Cherry Tree cottage, had been built, at least in part, during March 2000. The construction of the wall, shown in Fig. 3(b), comprised an upper course (0.5-m high and 2.0-m long stretcher baskets) placed above two courses of 1.0-m high and 2.0-m long baskets. The earth foundation that was supporting the gabion wall had settled in parts by up to 1.0 m. The back of the landslip was defined by the rear scarps that were evident in the roadway (Fig. 3c), the vertical scarp that was located immediately below the gabion wall and by the lateral extent of the distress in the roadway. No distress was evident to the northern part of the roadway or to the chapel or cottage buildings. Historical maps indicated that these buildings had already been in place prior to 1900.

2. Sequence of Events

The only reasonably comprehensive records that detailed the development of the landslip were the recollections of the occupier of the Cherry Tree cottage. The following sequence of events are based on these recollections. On the 5th of November 2000, a single kerbstone along the southern verge, and located directly in front of the cottage, was reported to have dropped overnight such that the top of the kerbstone had become level with the road surface. The next day the field below the embankment was flooded. Over the next two weeks, cracks began to appear parallel to the kerb along the road verge in the vicinity of the cottage. After several months of gradual downward movement, the gabions began to move outwards and later the gabion baskets in front of the cottage began to separate. The embankment slope next to the chapel building was reported to have begun moving shortly after the area in front of the cottage. In March 2002, the water flow from a discharge pipe located at the western end of the gabion wall ceased (Fig. 1). Gradual but progressive settlement was recorded, especially after periods of heavy rainfall, from the commencement of Scott Wilson’s involvement in the ground investigation in May 2002.

3. Desk Study

3.1. Geological Maps and Memoirs

Geological maps and memoirs (Aitkinhead et al., 1985; British Geological Survey) indicate that the site was situated on landslip debris that overlies undifferentiated mudstone and siltstone of the Millstone Grit Series. The geological maps show a narrow outcrop of Ashover Grit of the Millstone Grit Series immediately south of the site and running along the line of the 182-mAOD contour. This outcrop is shown to extend northwards beneath the site where it is obscured by the overlying landslip debris. A variety of mining related passages and shafts are also shown in the area although none were shown to be located closer than 150 m to the site. The effects of these features on the hydrogeology was uncertain.

3.2. Rainfall Data

The rainfall data recorded at the Middleton weather station for the period from mid October 2000 (before the November 2000 landslip) to May 2002 are shown in Fig. 4. The Middleton weather station (National Grid Reference SK25 275557) is located at an elevation of 321 mAOD (140 m above that of the Stanton Lees slip) and about 7 km to the south of the landslip site. As the weather station was remote from the slip, it is likely that the rainfall intensity values for the two sites would differ slightly over short periods. However, the overall rainfall trend for the sites would be expected to be broadly similar. The daily rainfall data recorded around the reactivation period of the slip in November 2000 are shown in Fig. 4a. The monthly rainfall data for the period leading up to the February 2002 slip are shown in Fig. 4b.

Heavy rainfall preceded the 5th of November event. October 2000 had produced 209% of the mean monthly rainfall, determined from recordings for that month over the preceding 30 years. On the 5th of November 2000, the recorded rainfall was 47 mm at the Middleton weather station, and another 28 mm of rainfall was recorded the next day. Autumn 2000 had also been the wettest season on record since local records began in 1870. Moreover, the recorded rainfall for the month of October 2000 was the second wettest since 1870 (the wettest being in 1998), while November 2000 had been the wettest on record since 1951.

4. Survey Data

4.1. General

The movement of the unstable ground and the level of the groundwater were monitored over a period of 14 months leading up to the commencement of the remedial works. Twenty five survey points (Hilti nails that had been driven into the tarmac) were installed along the roadway between the 13th of September and the 23rd of November 2001. Four survey lines were also set up over the embankment slope and the hillside directly below on the 25th April 2002. The survey lines each comprised seven or eight wooden pegs aligned at right angles to the existing gabion wall. Another two survey lines were installed on either side of the landslip to identify the lateral extent of its movement.

4.2. Survey Nails

Figure 5 shows the recorded movements in front of the Cherry Tree cottage. The most substantial movements corresponded to the survey points on the gabion wall and the area of sunken roadway at the western part of the slip. Half of the movement occurred between the 11th of January and the 5th of March 2002, which coincided with the high rainfall that had been recorded at the Middleton weather station (200 mm rainfall recorded for the month of February alone). The survey points that were located on the northern part of the roadway showed no significant signs of vertical or horizontal movements, with all of the recorded movements well within the survey measurement accuracy of ±5 mm.

4.3. Survey Pegs

Several of the pegs that were located on the embankment had moved down-slope by up to 80 mm indicating an underlying trend related to the developing landslip (Fig. 6). The greatest down-slope movements were recorded along the embankment crest.

5. Ground investigation

5.1. Stratigraphy

In total, seven trial pits and 19 boreholes were formed to depths of up to 13.6 mbgl through the main landslip during the course of three separate ground investigations at the site in 1999, 2001 and 2002. The 2001 and 2002 investigations were conducted by Derbyshire County Council Consulting Engineers and Scott Wilson, respectively, in order to identify the causes of the landslip. The boreholes were progressed below rockhead during the 2002 investigation. An air-flush, rock-roller bit (triple-tube core barrel) was used to obtain the rock cores, 70-mm in diameter. The core recovery was generally poor throughout, most likely due to either bands of more advanced weathering and/or more intense fracturing. The shallower boreholes were advanced using an automated, window sampling device in cases where undisturbed specimens were not required for geotechnical laboratory testing. Table 1 presents a summary of the different strata encountered at the site during the course of the ground investigations.

The embankment comprised very soft to firm fill and Head materials which had been softened locally by groundwater seepage from the underlying sandstone and mudstone bedrocks. The joints within the recovered sandstone cores were in a damp condition. Core material recovered next to the zones of no recovery in the mudstone and siltstone bedrocks were in a wet state. In some of the trial pits, minor shear surfaces of limited persistence were recorded immediately below the interface between the Head material and the underlying completely weathered mudstone/siltstone.

5.2. Groundwater Conditions

Standpipe piezometers were installed in all of the boreholes that had been drilled during the 2002 ground investigation. A string of piezometer buckets (plastic containers with perforated ends) were located at 0.5-m centres over the anticipated range of groundwater levels. Lead weights were secured to the bottom of the string to maintained the position of the buckets in the borehole as the level of the groundwater increased. The presence of groundwater was recorded in all of the instruments. Seepage from an abandoned water main (cast iron pipe, 75 mm in diameter) located beneath the northern road verge may also have caused localised softening of the embankment fill material although five trenches which had been excavated across the roadway to locate services revealed no damage to the pipe. Overall, the seepage volumes from the abandoned pipe would have been minor in comparison to the groundwater flow during the heavy rainfall events leading up to the reactivations of the slip.

6. Stability Back Analysis

6.1. Geological Profile

The ground investigation indicated that the most adverse ground conditions as regards slope instability were through the embankment slope located directly in front of the Cherry Tree cottage (Fig. 7). This section of embankment had the steepest slope; the largest recorded wall deformation and the greatest thickness of superficial deposits overlying bedrock. The gradual but progressive slope movements indicated that the embankment itself was at limiting equilibrium.

6.2. Soil Parameters

The soil parameter values used in the preliminary stability back-analysis were obtained from the ground investigation and geotechnical laboratory test data, and indirectly from published correlations. The peak and residual values of the effective stress shear strength parameters were measured using the consolidated-undrained triaxial compression apparatus, including measurement of pore water pressure response, and the Bromhead ring shear apparatus, respectively. Site observations and the findings of the ground investigation indicated that the landslip had occurred due to the superficial deposits moving over the weathered bedrock, and as such, the weathered bedrock was simply assigned notional and conservative (high) parameter values in order to ensure that this material remained static during the slope stability analysis.

The levels of the groundwater that had been recorded behind and down-slope of the existing gabion wall were used in modelling the groundwater profile. It was doubtful however that the standpipe piezometers had been monitored over a sufficiently long period of time to determine the highest groundwater profile that existed around the reactivation of the slip. Hence, an additional 1.0 m head of water was added to the highest recorded groundwater levels to simulate the effects of the rising groundwater in the slope stability analysis and to account for other adverse conditions (including possible seepage from the abandoned cast iron, water main located beneath the road). A sensitivity study showed that higher groundwater levels would have required proportionate increases in the values of the shear strength parameters to sustain limiting equilibrium and, as such, would have lead to an unconservative design for the remedial works (values of the shear strength parameters higher than the actually values).

6.3. BACK ANALYSIS

A stability back-analysis of the embankment slope was carried out using the *Slope/W* software developed by Geo-Slope International Limited. In the ground model (Fig. 8), the back scarps day-lighted in the roadway and the shallow rotational slips (recorded onsite in the embankment fill material) day-lighted in front of the gabion wall. A factor of safety (FOS) value of unity, consistent with the gradual progressive slope movements, was obtained using the Morgernstern-Price method (which provided the most rigorous analysis) and the input parameter values listed in Table 2.

A sensitivity study, which considered the affect of minor inaccuracies likely in the measured surface and geological profiles, was carried out to ensure that the values of the effective stress shear strength parameters were realistic. The reduction in the FOS value that accompanied a 1.0 m increase in the highest recorded groundwater level (as might have occurred at the time of the November 2000 slip) indicated that the overall stability of the embankment was particularly sensitive to fairly minor changes in the groundwater levels. The stratum of Head material further down-slope of the embankment toe was stable (FOS $\cong$ 1.3) under the worst groundwater conditions recorded (adopting peak values of the shear strength parameters in the analysis since no slips had been recorded in the natural hillside).

It was not necessary to invoke the residual values of the shear strength parameters (measured using the Bromhead ring-shear apparatus) for the completely weathered mudstone and Head material since the stability analysis indicated that the embankment slope was at the limiting equilibrium condition for the peak parameters values. A reduced angle of shearing resistance (ϕ') value of 26° for the Head material (failed condition) gave a FOS value of unity for the shallow translational slips day lighting in the roadway and directly below the gabion wall foundation. However, lower ϕ' values could not have occurred for the Head material over an extensive area of the slip surface since such a condition would have resulted in a catastrophic failure having occurred.

6.4. INFERRED FAILURE MODE

The embankment slip occurred when the groundwater levels had risen following a prolonged period of heavy rainfall. The downward movement of the embankment slope undermined the foundations of the existing gabion wall which resulted in its outward and downward movement, and ultimately the partial collapse of the roadway. The shallow rotational slips day lighted directly beneath the wall foundations. An additional load was placed on the head of the slip following the forward movement of the wall and its backfill. It is likely that the tilting of the wall also exaggerated the eccentricity of the loading on the foundations, which contributed to the local bearing capacity failures that occurred.

7. Remedial Works

7.1. REMEDIAL OPTIONS

The remedial works adopted necessitated the construction of a new cantilevered, bored-pile retaining wall in place of the gabion wall, and with all of the construction works occurring from the road level. Variations on the retaining wall theme that would have involved the construction of the new wall at the mid-height or near the toe of the embankment slope were also studied. However, the practical difficulties of carrying out the temporary works on the slope face (which could have potentially caused its progressive destabilisation) were considered too difficult to overcome. Regrading and counterfort drainage of the embankment slope were also ruled out for the same reasons. An anchored, bored pile wall aligned along the road verge was also considered but ruled out due to concerns about the installation of the ground anchors (required lengths of 8 to 10 m) beneath the privately owned Cherry Tree cottage and its gardens located next to the site (Figs. 1 and 2).

7.2. Bored Pile Retaining Wall Solution

The design solution adopted necessitated two rows of cantilevered bored piles (86 in total and each 600 mm in diameter) installed along the road verge (Fig. 9). The location of the piles were staggered in plan arrangement, with a centre spacing of between 2.0 and 2.7 times the pile diameter between adjacent piles. This configuration had the added benefit of allowing the easy passage of groundwater, as compared to a contiguous wall solution.

The staggered pile wall was analysed for stability using the GEOSOLVE *WALLAP* programme (version 5.01) on the basis of a one-metre strip (plane strain) and the Burland-Potts method. The new wall was assumed contiguous for the purposes of the analysis since the pile centre spacing was less than three times the pile diameter. Portal frame action between the two rows of piles was not considered in the analysis since they were so closely spaced. The pile toes were founded in the underlying mudstone and siltstone bedrocks.

The worst possible case of slippage occurred directly in front of the Cherry Tree cottage, and based on the ground investigation data, the thickness of the Head material that was overlying the weathered sandstone/mudstone/siltstone was conservatively assumed to be 6.0 m. The progressive down-slope movement of the relatively steeply sloping ground would ultimately cause a loss in passive support to the new wall over the 6.0 m depth of Head material. Hence, no passive resistance was assumed to act over this depth in the analysis. The wall design shown in Fig. 10 assumed that the failed groundmass on the passive side would only provide a surcharge (in the form of a trapezoidal stress distribution) to the underlying weathered rock. A live surcharge loading of 5 kN/m^2 (as specified by Derbyshire County Council Consulting Engineers) was also applied across the roadway in the analysis. The Young's modulus and moment of inertia values of the cantilevered pile wall were 2.0×10^7 kN/m^2 and 0.011 m^4/m-run, respectively.

The groundwater levels used in the design were based on the highest recorded groundwater levels. An additional 1.5 m head of water was applied behind the new retaining wall. However, only a 0.5 m head difference across the wall (to cover for possible build-ups of flush water used in the piling works) was considered for the temporary (short-term) condition check.

Two main factors of safety were considered in the geotechnical stability and structural design calculations, namely:

1. For geotechnical stability; FOS $\geq$ 2.0 on the net available passive earth pressure. Apart from the embankment fill material, the values of the earth pressure coefficients included a contribution due to the interface resistance mobilised between the soil and the concrete

piles, with $\delta/\phi = 0.66$. The values of the Rankine earth pressure coefficients were used in the case of the embankment fill material.

2. For structural design; FOS $\geq$ 1.65 on the earth pressures. The design bending moments and shear forces for the ultimate limit state design of the structural members were calculated by the *WALLAP* programme using the subgrade reaction model and the values of the earth pressure coefficients for the 'at rest' condition.

The pile embedment depths were increased above the values calculated by the *WALLAP* analysis in order to allow for the adverse combination of sub-vertical and sub-horizontal fracturing in the bedrock, with the final overall pile lengths ranging between 8.0 and 13.5 m. A reinforced-concrete capping beam was also poured (Fig. 9b) in order to connect the pile heads; enhance the wall integrity and even out the lateral displacements of the pile rows. A mortared, gritstone parapet wall was constructed above the capping beam. Samples of the Head and embankment fill materials were tested to determine their aggressiveness to concrete. pH values of 7.2 to 7.3 and a soluble sulphate content of 0.19 to 0.26%SO_4 were measured for a 2:1 water/soil extract. A soluble sulphate value of 0.26%SO_4 (i.e. 2.6 g/l SO_4) requires a design sulphate class DS-3 in accordance with BRE (2001). An ACEC class AC-3 concrete was specified for the characteristic pH value of 7.2 and the groundwater conditions encountered onsite. Grade 35N concrete was used in the construction of the bored piles.

7.3. Construction works

The engineering works were carried out by Piling Solutions, UK. The 600-mm diameter cavities for the piles were drilled using a compact, 25-tonne Bauer rig (Fig. 9c). The unsuitable subgrade material that was located beneath the affected roadway was excavated and replaced with granular fill (thereby providing free sub-surface drainage behind the retaining wall) and the road pavement was reconstructed over its full depth. Further details on the construction works have been reported by Forrest (2003).

8. Summary and Conclusions

The reactivation of the landslip near the village of Stanton Lees in November 2000 resulted in the gradual but progressive down-slope movement of an earth embankment that had supported an existing gabion retaining wall and a minor roadway. The site was located on relatively steeply sloping superficial deposits which overlaid undifferentiated mudstone and siltstone strata of the Millstone Grit Series. Site observations and the findings of the ground investigation indicated that the landslip had occurred due to the superficial deposits moving over the weathered bedrock when the groundwater levels were elevated following a prolonged period of heavy rainfall; autumn 2000 had been the wettest season on record in that locality.

A slope stability analysis indicated that the embankment was at limiting equilibrium and that the factor of safety on slope instability was particularly sensitive to fairly minor changes in the groundwater levels. All of the remedial works were completed from road level since practical difficulties in establishing machine plant or carrying out temporary works on the slope face were considered too difficult to overcome. The final design solution adopted required the construction of a cantilevered, bored pile retaining wall along the road verge in place of the existing gabion wall. The new wall comprised two rows of 600-mm diameter piles (staggered in plan arrangement) with the pile toes founded in the underlying mudstone and siltstone bedrocks. The pile heads were connected together using a reinforced-concrete capping beam to enhance the wall integrity.

Acknowledgements

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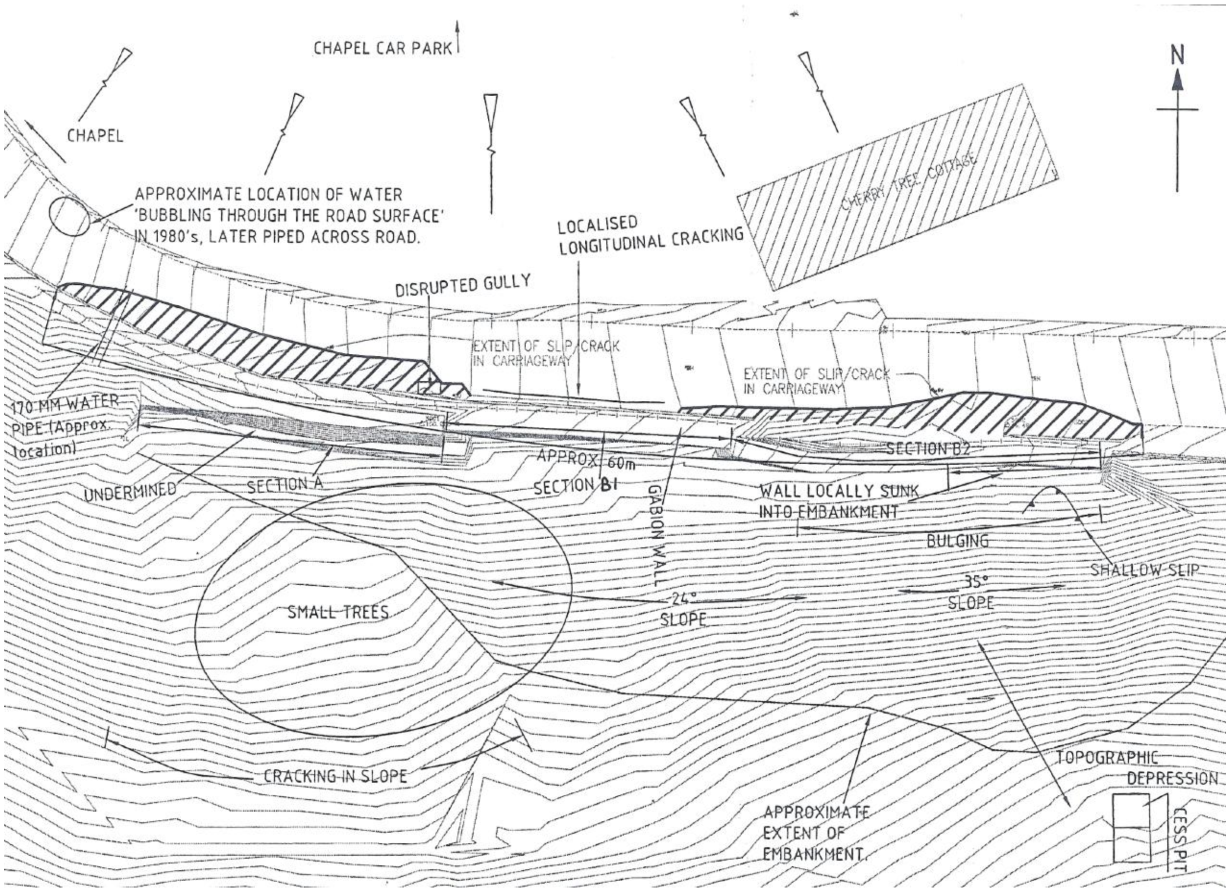


Figure 1. Main features of the landslip.

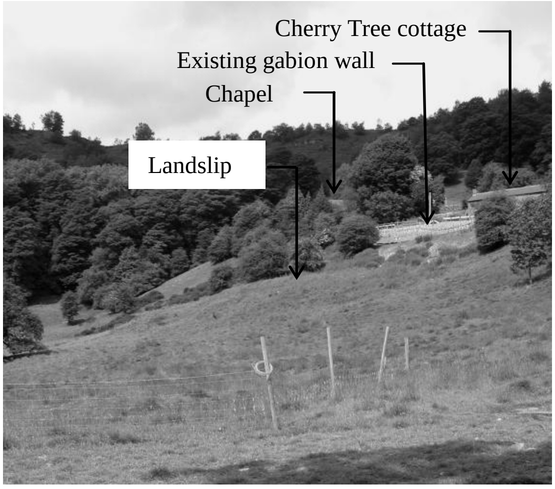
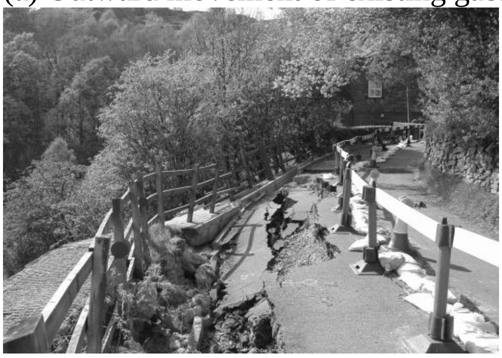


Figure 2. Site topography viewed towards the upslope direction.

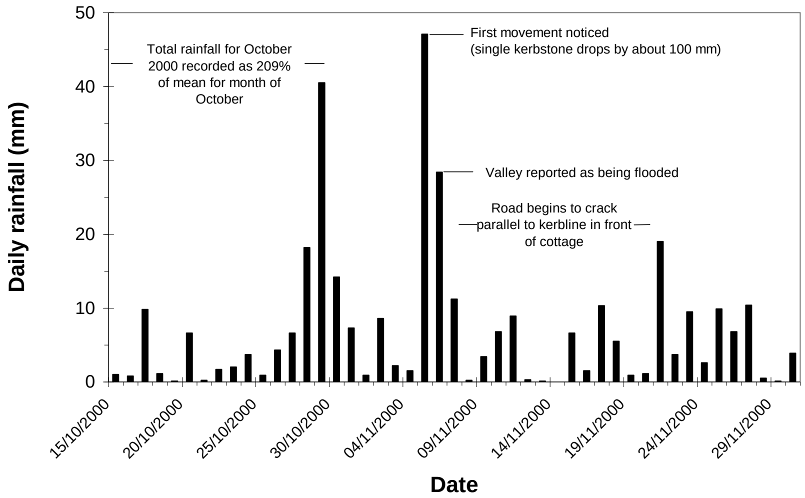
(a) Outward movement of existing gabion wall.

(b) Settlement of gabion wall foundations.

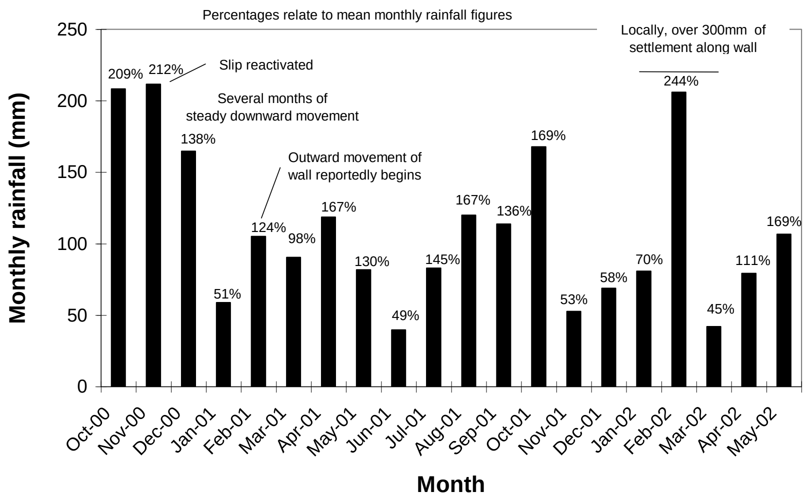


(c) Collapse of southern part of road.

Figure 3. The Stanton Lees landslip.



(a) Daily rainfall data leading up to the November 2000 slip.



(b) Monthly rainfall data for the period covering the November 2000 and February 2002 slips.

Figure 4. Rainfall intensity recorded at Middleton, UK.

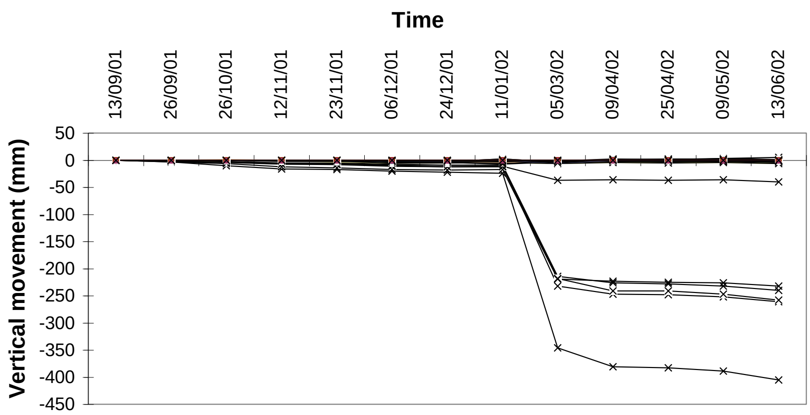


Figure 5. Vertical movements of survey points along roadway.

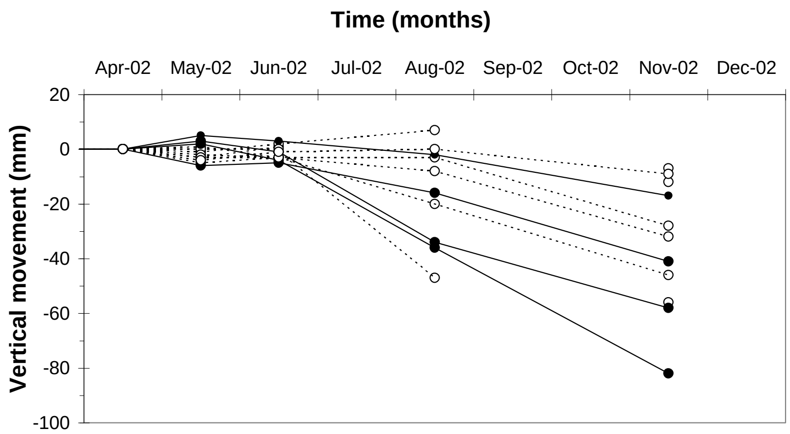


Figure 6. Vertical movement of survey pegs on the embankment slope. Note that the solid lines and symbols denote pegs located along embankment crest.

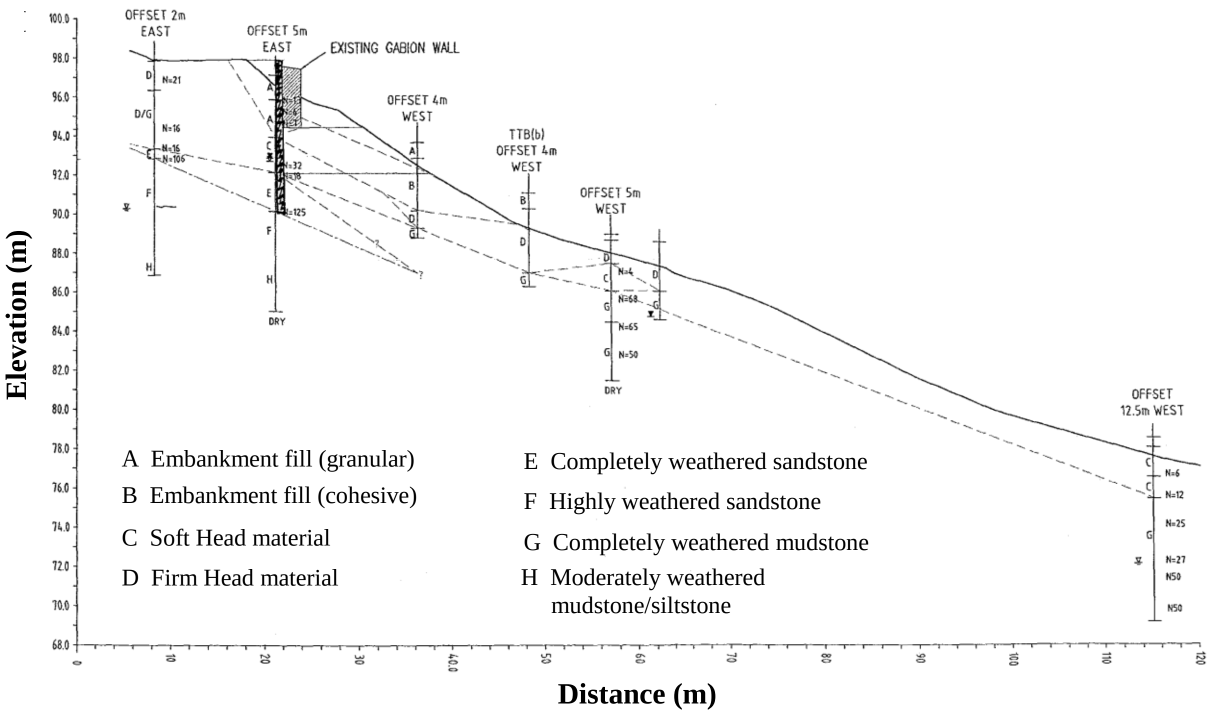


Figure 7. Geological cross-section of the site. Note that the elevation levels relate to a temporary datum and local grid set up near the chapel building.

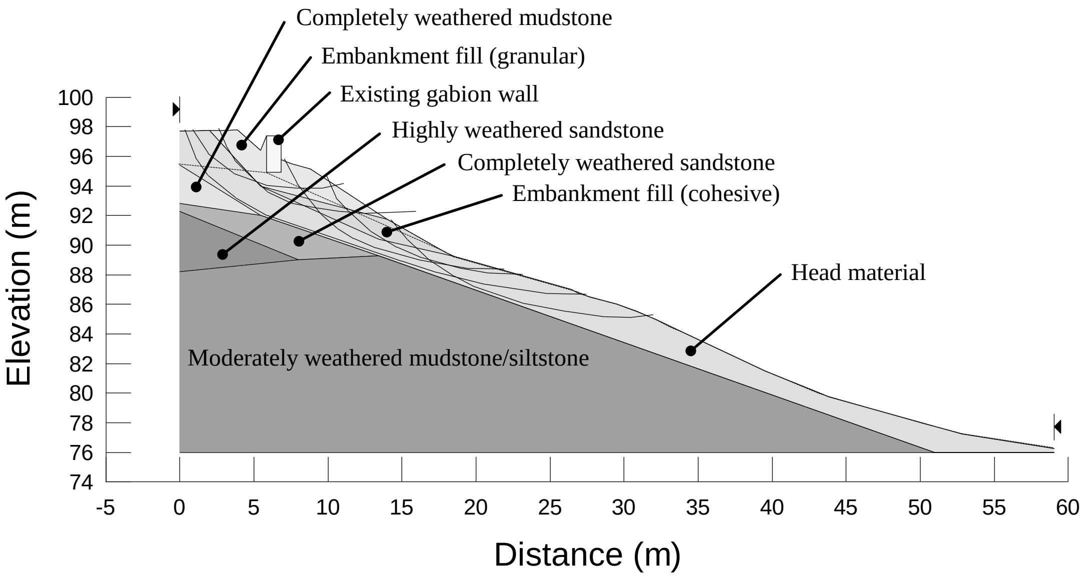
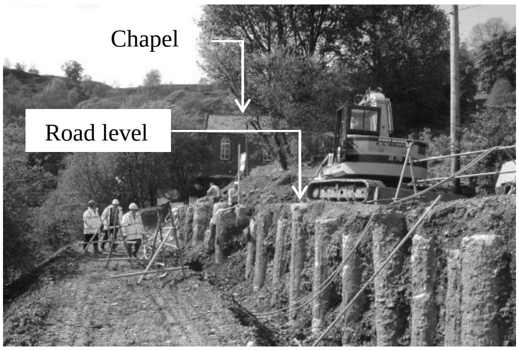


Figure 8. Slope/W analysis. Note that the elevation levels relate to a temporary datum and local grid set up near the chapel building.



(a) Outermost row of piles in place.

(b) Placement of formwork for capping beam.

(c) Lightweight augur rig.

Figure 9. Construction of cantilevered, bored pile retaining wall.

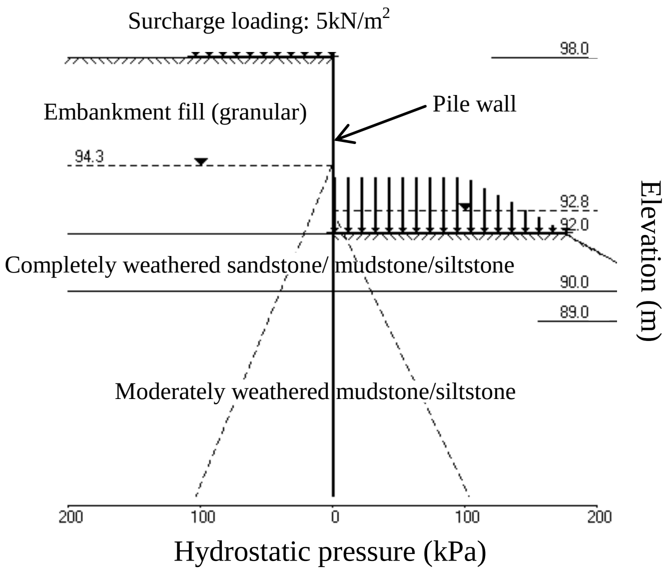


Figure 10. Design model for *WALLAP* analysis.

Embankment fill comprising loose and medium dense, sandy gravel inter bedded with soft and firm, sandy, gravelly clay and very silty clay of intermediate plasticity.
Head material comprising very soft to firm, sandy, gravelly clay of intermediate plasticity, with cobbles and occasional boulders.
Firm-to-stiff, fissured, laminated clay (completely weathered mudstone)
Medium dense to very dense, highly to completely weathered sandstone
Weak to moderately weak, closely to medium jointed, moderately weathered mudstone/siltstone.

Table 1. Stratigraphy at the Stanton Lees slip.

Stratum	**Bulk unit weight (kN/m^3)**	**Effective stress parameters**	
		c' (kPa)	**∅' (degrees)**
Embankment fill (granular)	17	0	32
Embankment fill (cohesive)	18	0	30
Head material (peak) (residual)	19 19	0 0	30 12
Highly to completely weathered sandstone	20	0	34
Completely weathered mudstone (peak)	21	5	30
Moderately weathered mudstone/siltstone	23	50	45

Table 2. Soil parameter values for the limiting equilibrium condition.