SLOPE STABILISATION OF A SECTION THE THIRLMERE AQUEDUCT BY THE USE OF SPACED MICROPIES AND PERMEATION GROUTING

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SYNOPSIS

The Thirlmere aqueduct was constructed over 110 years ago to provide water from the Lake District (in the north-west of England) to the city of Manchester some 60 miles away. The aqueduct remains, to this day, a vital piece of infrastructure in the distribution of raw water in the north-west of England.

A short section of the aqueduct was identified as being in poor condition during routine inspections by the owner. Subsequent investigations attributed the conduit damage to the movement of unstable ground over the conduit. Bachy Soletanche Ltd was subsequently appointed to carry out the design and construction of the slope stabilisation works and to undertake repairs inside the damaged conduit. The works were undertaken on an existing slope which, in places stands at a gradient of up to 40º to the horizontal. Strict limitations on plant and equipment were imposed by the client to ensure that slope and conduit loading and associated vibrations were kept to an absolute minimum during construction. An intensive instrumentation system was installed to monitor surface movements, ground movements at depth, conduit movement and strains across existing planes of weaknesses within the conduit. The solution comprised the installation of spaced piles up-slope and down-slope of the conduit. Restrictions were imposed on the minimum stand-off distance between the outside face of the conduit and the piling works so targeted permeation grouting was undertaken to improve the ‘connection’ between the conduit and the down-slope piles. The presence of existing sub conduit groundwater flows was suspected and the permeation grouting was focused in areas to minimize disruption to the groundwater regime.

The paper presents the basis of the design and highlights the combined construction and instrumentation methods adopted to minimise damage to this valuable asset.

INTRODUCTION

The Thirlmere Aqueduct supplies water from the Lake District to Manchester. It was constructed in the 19th century and consists of single line tunnel, conduit sections and large diameter pipe sections. Pipe siphons convey the aqueduct across numerous valleys, Hill (1896).

Nab Scar conduit is a short length of aqueduct conduit linking two aqueduct rock tunnels. It is situated on a steep hillside above Rydal Village, north of Ambleside. A photograph taken during the construction of a rock tunnel portal in the area of Nab Scar is shown in Figure 1 and view of the existing Nab Scar hillside is shown in Figure 2.

The purpose of the project was to stabilise the conduit section of the Thirlmere Aqueduct at Nab Scar and to undertake internal concrete repairs during a 4 week Outage period during the autumn of 2009.

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Figure 1. Rock tunnel portal during construction near Nab Scar
Source United Utilities web site

Figure 2. Looking towards Nab Scar, with approximate position of conduit shown

HISTORY

The 50m long section of Thirlmere Aqueduct at Nab Scar extends, in conduit, between two sections of rock tunnel. This section of conduit runs in a general south-easterly direction, before turning left through an angle of 55°, to enter the Nab Scar Tunnel portal. Desk study investigations suggest that the conduit was originally built across a natural gulley in the hillside, and was then covered over with rock fill on completion. Gallagher et al (2009).

The conduit is a mass unreinforced concrete structure with continuous vertical walls and arched roof and an infill concrete floor slab. There is no structural connection between the walls of the conduit and floor slab and the construction joint is able to rotate and displace. See Figure 3.

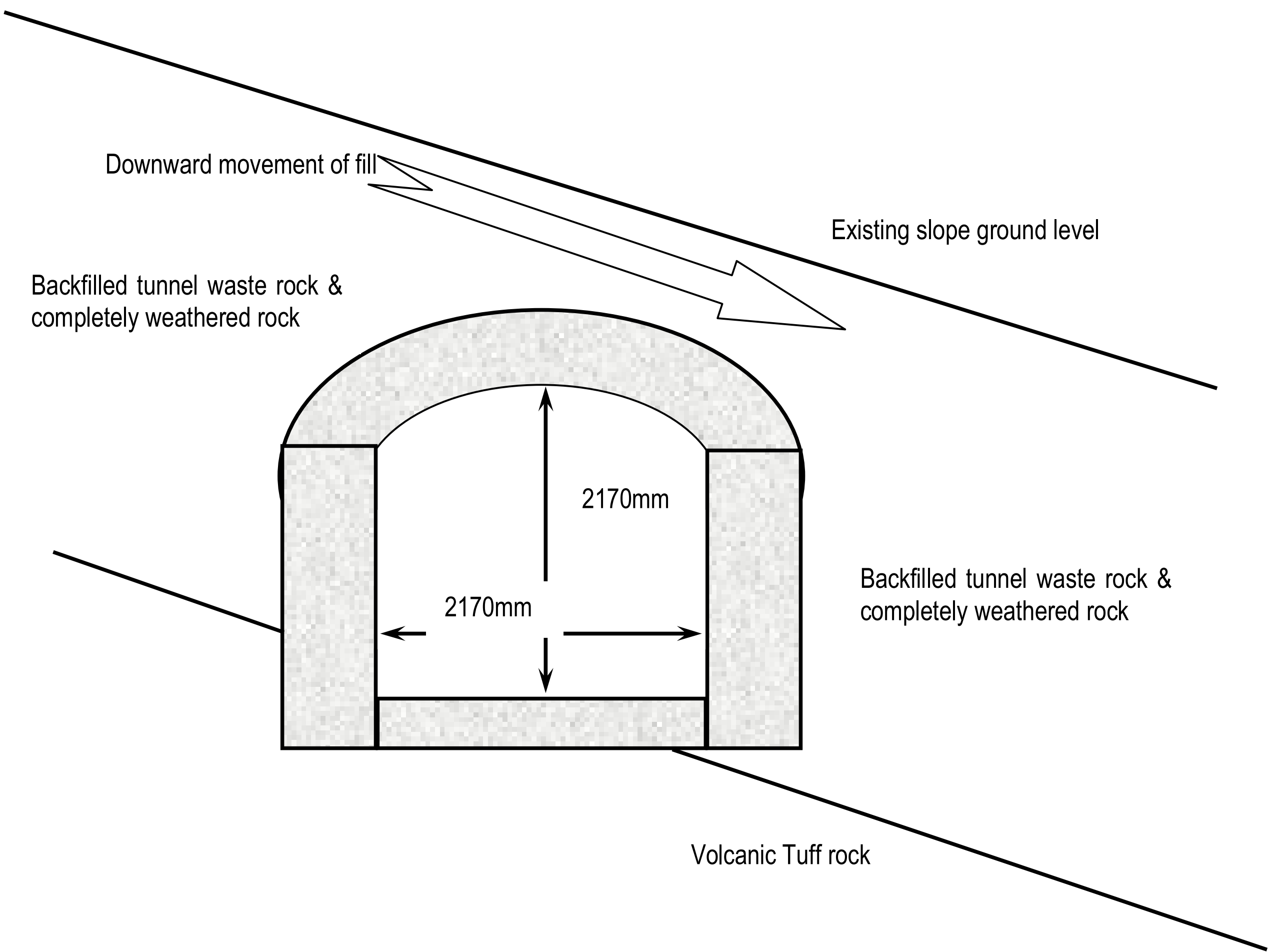

Figure 3. Typical cross section through mass concrete conduit at Nab Scar

A series of cracks were evident within the conduit structure and the most marked of them comprised of a spiral crack (Figure 4), which ran from the wall/floor joint on the outside of the bend (right hand side) in the conduit and around the barrel of the arch, terminating at the tunnel portal. The crack was first noted in 2005 during the first outage in the current programme. Remote reading movement gauges were installed across significant cracks to provide early warning of any future movement. The gauges are read frequently and, to date, have not shown any further movement. However, the removal of the manganese deposits, as part of previous internal repair works, revealed other cracks, which were not visible to the 2005 inspectors, particularly an opening of the joint between the base and the wall on the down-slope side of the conduit. The orientation of the cracking, together with the opening of the base wall joint, indicates that the conduit structure has undergone a torsional movement, with the downstream section twisting outwards (down-slope) relative to the upstream section.

Figure 4: Photograph taken inside of the conduit showing spiral cracking

During subsequent outages, a series of surveys and intrusive investigations has established that the floor of the conduit is founded directly on a platform of rock and so the potential for global movement of the conduit was discounted. Instead, it is considered that the displacement is limited to a section of outer wall and roof on the outside of the spiral crack.

It is understood that the conduit at Nab Scar was constructed and then buried beneath waste materials from the tunnelling operations. The tunnelling waste primarily comprises cobbles and boulders of tuff, within a secondary matrix of sand and gravel. It appears that the waste material was end tipped within the gully and so stands at or close to its natural angle of repose. There is no evidence to suggest that the material was compacted and there are clear indications that that waste on the down-slope side of the conduit has slumped downhill, potentially removing lateral support from the outside of conduit structure. Waste material on the upslope side of the conduit also shows signs of movement and the passage of material over the conduit would drag across the roof of the structure. Accordingly, it was concluded in the report by Gallagher et al (2009) that the torsional displacement of the conduit was caused by external forces due to movement of the overlying fill material, as shown schematically on Figure 3.

REQUIREMENTS AND RESTRICTIONS

In order to protect the conduit from the effects of further movement of the tunnelling waste, reinstatement of lateral support was required to the outside wall of the conduit. A secondary form of protection was also required to shield the upslope side of the conduit from lateral pressure due to movement of the upslope body of ground/tunnelling waste. The investigations had revealed the presence of significant voids within the tunnelling waste material, and the proposed solution was required to reduce these voids. Restrictions on drilling close to the conduit were imposed to minimise the potential for damaging the weak structure. No works were allowed to be undertaken within 2m of the outside face of the conduit.

The following restrictions were imposed by the client to protect the conduit from any further damage:

- No direct loading of conduit;
- Construction methods had to adopt techniques that imposed minimal vibrations on to the conduit, the use of percussive drilling techniques and down the hole hammers were expressly prohibited;
- All slope loading to be tied back to stable rock outcrops;
- Conduit and slope movements to be monitored through out the works with works ceasing if trigger levels reached.

GROUND CONDITIONS

The ground conditions comprised a variable thickness of Made Ground of up to 5m thickness over completely weathered rock (up to 3m thickness) over moderately strong Volcanic Tuff. See Figure 3.
Groundwater levels were monitored within standpipe piezometers installed up-slope and down-slope of the conduit. Groundwater was not encountered during the drilling of any of the site investigation boreholes but was measured at depths of 0.72m to 1.1m below existing ground levels in piezometers installed within the Made Ground. Piezometers sealed within the rock recorded groundwater levels in the range of 2.1m to 7.6m below ground levels. The difference in groundwater levels recorded within the shallow and deep piezometers indicates the upper groundwater to be perched within the Made Ground. The groundwater monitoring was undertaken over a period of less than one year. Observations of the standpipes during inclement weather did not show groundwater rising to the surface of the slope. The apparent groundwater gradients within the Made Ground and rock were broadly parallel to the slope and their levels varied between the ranges stated above depending on preceding rainfall.

MONITORING AND INSTRUMENTATION

An instrumentation scheme was developed to monitor the movements on and within the conduit and to measure movements of the hillside during thr construction phase. The instrumentation comprised of the following:

<u>Hillside surface monitoring</u>

A fully automated total station surveying system was employed to continuously monitor hillside ground movements in real-time. The system is a fully automated, optical monitoring system comprising of a motorised total station that observes prismatic targets fixed to the hillside. The monitoring prisms were installed on a square matrix with prisms at 5.0m centres across the hillside. Photographs of the system are provided in Figure 5.
The total station unit was piloted by a single industrial PC which operates the instrument, logs the results, undertakes the survey calculations, performs quality control checks and then internally stores each prism's observed co-ordinates. The PC automatically downloads the acquired survey co-ordinate data and imports the information into the monitoring database for subsequent analysis.

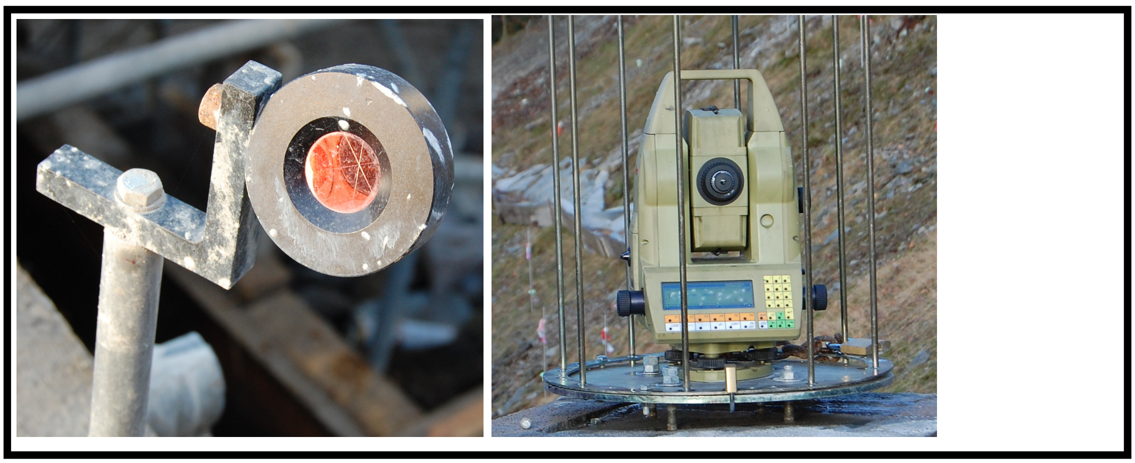

Figure 5. Automated monitoring system – prism and total station

Conduit monitoring

Five survey targets were fixed to the external surface of the conduit's roof structure to enable movement of the structure to be monitored whist the conduit is in operation.

Hillside ground monitoring

Five inclinometers were installed across the hillside. The inclinometer boreholes were between 4.0m and 6.0m deep and were socketed into competent rock by a minimum length of 1.5m to provide a suitable point of fixity. ABS inclinometer casing, 70mm in diameter was fixed within the borehole using cement grout. Manual baseline readings were taken before connecting the in-place-inclinometers to the site data logger. The in-place-inclinometer sensors automatically recorded lateral displacement of the slope at hourly intervals and reported the data to the database.

Internal monitoring of the conduit

Ten strain gauges were fitted within the conduit across existing cracks within the conduit. The gauges were bonded to the inside of the conduit and surrounded in a stainless steel shield. The space inside the shield was filled with silicone sealant.

The automatic strain gauges recorded stain levels on an hourly basis and feed the data back to the instrumentation database.

<u>Instrumentation database</u>

All of the instrumentation data was stored on the database. The database enables rapid interrogation and manipulation of the data to provide useful information. The database was programmed with pre-determined trigger levels for the different instruments. A warning beacon was established on site to warn site workers of movement and automated SMS messages were sent members of the team when trigger levels were reached.

SELECTED SOLUTION

The selected solution was to use spaced piles above and below the conduit to stabilise the hillside and conduit. Direct structural connection to the conduit was not permitted and so permeation grouting via a grid of TaM grouting boreholes were installed between the conduit and the down-slope spaced piles. The permeation grouting would back-fill open voids and to provide an indirect structural connection between down-slope piles and the conduit. The permeation grouting was designed to be installed in isolated blocks to enable the passage of existing groundwater flows to continue unhindered. The works could be undertaken with light weight drilling rigs working off a temporary scaffold platform, which itself, was anchored back to existing stable rock outcrops. See Figure 6.

Figure 6. Light-weight drilling operating of temporary scaffold platform

DESIGN OF SPACED PILES

General

The selected slope stabilisation design comprised the use of two rows of spaced piles. One row of spaced piles being provided up-slope of the conduit and one row of spaced piles being provided down slope of the conduit. Each group of spaced piles comprised of two micropiles as follows:

- one vertical micropile acting in compression
- one raked micropile acting in tension which will be raked by 40 degrees from horizontal pointed towards the slope.

The micropiles were provided in pairs, tied at their head with a pile cap. The pairs of micropiles were termed "A-frames". The A-frames act as a passive restraint system whose resistance is mobilized following minor downward movement of the ground up-slope of the piled A-frames. A typical section through of the design is shown in Figure 7.

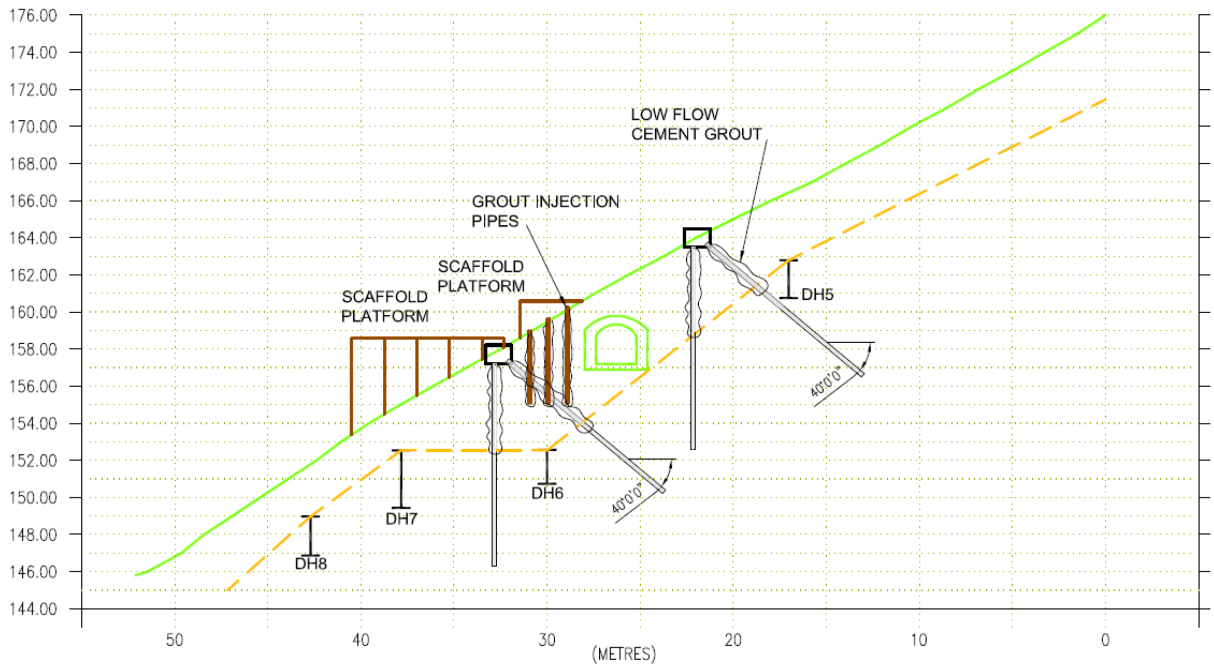

Figure 7. Typical section through slope stabilisation works

Slope stability

The slope, pre-stabilisation, was considered to have a low factor of safety against failure (approaching unity). Back analysis calculations of the slope where undertaken considering a factor of safety of 1 to back calculate soil parameters for the previously determined geometry and groundwater conditions. The back-calculated parameters are provided in the calculations shown in Figure 8 and Figure 9. Slope stability calculations were undertaken using the slope stability software package 'SLOPE' developed by Geosolve. Circular and non-circular slip failure mechanisms were considered. Following the determination of the back-calculated soil parameters, critical slope failure slips were repeated with inputted restoring forces to establish the required restoring resistance to provide an engineered slope with a factor of safety of at least 1.3. The required up-hill restoring force was calculated as 250kN per m length of slope, acting in a direction parallel to the slope.

Infinite slope stability calculations were also undertaken to provide comparison with the SLOPE software results. These calculations provided good agreement with the computed factors of safety for the existing condition (FoS = 1.0) and the strengthened condition (FoS =1.3). See Figures 8 and 9 below. A sensitivity analysis was undertaken to consider different slip depths and different angles of frictional for the Made Ground/weathered bedrock.

The factor of safety, F, against translation slip failure is given by the equation below for an infinite slope with seepage parallel to ground surface and where ground water is not at the surface of slope

$$F = [c/(\gamma_{sat}.z.\sin\beta.\cos\beta)]+[(\gamma_{sat}-\gamma_w.m)/\gamma_{sat}].[\tan\phi/\tan\beta]$$

where:

saturated soil unit weight	γ_{sat}	20	kN/m^3	
unit weight of water	γ_w	10	kN/m^3	
angle of friction	ϕ	45	º	45
slope angle to horizontal	β	34	º	
cohesion	c	0	kPa	
depth to slip surface, z		2.5	m	2.5
depth to groundwater surface, z_w		1	m	

$m = (z-z_w)/z$ 0.6

m = fraction from 0 to 1 (=1 for ground water at slope surface)

F = **1.04**

Figure 8. Infinite slope calculations

The infinite slope equation shown above can be re-written for finite slopes to allow incorporation of a restoring force term. The resulting equation is shown in Figure 9.

The micropile A-frames were designed to provide the calculated slope stabilising force and to thereby increase the slope factor of safety to at least 1.3. The up-slope A-frames were designed to provide the required restoring force to achieve a post stabilisation slope stability factor of safety of 1.3. The down slope row of A-frames resists significantly smaller forces than the up slope row – only the section of slope between the two rows of A-frames. Nevertheless identical piles, albeit at slightly wider spacings, were designed within the down slope row, to provide down-slope support to the conduit. The calculation of pile loads and associated pile deflections are discussed below.

The above equation can be re-written to include a restoring force H term, where H is the restoring force parallel to the slope in kN per m run of slope.
The inclusion of this term requires the slope length, l, being considered to be included in the calculation of the factor of safety, F.

The factor of safety, F, against translation slip failure is given by the equation below for a 'specific' slope with seepage parallel to ground surface and where ground water is not at the surface of the slope.

$$F = \frac{c.l}{\gamma_{sat}.l.z.\sin\beta.\cos\beta} + \frac{(\gamma_{sat}.l.z.\cos^2\beta - \gamma_{\omega}.m.z.l.\cos^2\beta).\tan\phi + H}{\gamma_{sat}.l.z.\sin\beta.\cos\beta}$$

saturated soil unit weight	γ_{sat}	20	kN/m^3	
unit weight of water	γ_w	10	kN/m^3	
angle of friction	ϕ	45	º	0.785398 radians
slope angle to horizontal	β	34	º	0.593412 radians
cohesion	c	0	kPa	
depth to slip surface, z		2.5	m	
depth to groundwater surface, z_w		1	m	
slope length, l		38	m (length to top of slope)	
Restoring force parallel to slope, H		250	kN per m	
$m = (z-z_w)/z$		0.6		

m = fraction from 0 to 1 (=1 for ground water at slope surface)

F = **1.32**

Figure 9. finite slope equation re-written to include slope restoring force term

Calculation of pile loads

The calculation of A-frame loads and deflections were calculated by two methods:

- Structural frame model
- Elastic continuum model

Both of these methods are reported as suitable methods by Elson (1984).

Structural frame model

The structural frame model is based on resolving the horizontal force into two components, producing an axial compressive force in the far pile and tensile force in the near pile. The restraint offered by the pile cap is ignored, and the magnitude of each component is obtained from a simple triangle of forces as shown on Figure 10. Estimates of deflections were made on the basis of elastic compression/extension of the pile section resisting the compression/tension forces.

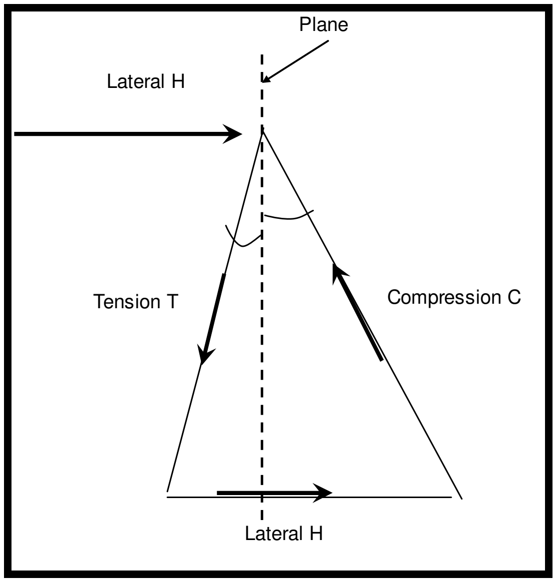

Figure 10. Triangle of forces used to estimate micropile axial loads

The slope restoring force of 250kN per m run of slope, acting parallel to the slope, was determined as the force required to increase the slope's factor of safety against slope failure to 1.3. This restoring force was to be provided by A-frames spaced at 2.0m centres and were therefore designed to carry safe working loads of 500kN. The second line of raking piles down-slope of the conduit were identical to the up-slope pairs other than they were spaced at slightly wider centres (2.25m) along the slope. The lower line of A-frames provided support to the conduit and are expected to be lightly loaded once the upslope piles have been installed and resistance mobilised. One of the reasons for maintaining a similar pile arrangement down-slope of the conduit was in order that stability could be built into the slope during the progression of the works. The client would not have permitted the erection of temporary scaffolding and operation of equipment up-slope of the conduit without stabilisation measures being in place down-slope of the conduit first.

In the long-term, the full safe working micropile load is only expected to be mobilised on the up-slope pairs of piles. The calculated lateral deflection for these piles, under the full safe working load, was estimated to be of the order of 10mm using the structural frame model. This model estimates deflections by ignoring any contribution from the soil. Lateral deflections were estimated by resolving the individual structural members' deflections under the action of the calculated loads assuming that the members behave elastically.

Elastic continuum model

The 'Piglet' software, published by Randolph, was been used to undertake the elastic continuum model calculations.

The pile group analyses to determine pile loads and group performance have been undertaken using Piglet version 5.1. The solution for laterally loaded pile groups adopted in the program is one developed by Randolph (1981a) by curve fitting the results of finite element analyses of laterally loaded piles embedded in elastic 'soil'. It was found that, for piles which behave flexibly under lateral load, simple power law relationships could be developed giving the lateral deflection, u, and the rotation, θ, of the pile at the soil surface, in terms of the pile stiffness and the soil properties. The relationships are similar in form to those arising from considering the soil as a Winkler material characterised by a coefficient of subgrade reaction (e.g. Reese and Matlock, 1956; Matlock and Reese, 1960). Horizontal loads and bending moments at the head of each pile may then be calculated from lateral deflection, u, and the rotation, θ, by equations that take account of the appropriate shear modulus of the soil, pile stiffness and the critical pile length (depth to which the pile deforms appreciably). Please refer to Randolph (2004) for a full account of the analysis method. The pile is taken to be fixed within the pile cap such that rotation of the pile cap is not permitted. This is reasonable given the penetration of the pile into the cap and the relative stiffness between the piles and the pile cap. The software then calculates the pile cap fixing moments to ensure zero rotation of the cap.

The Piglet input parameter values are given below:

- Pile layout: based on two pairs of 'A-frames'at 2.0m centres for the up-slope piles and 2.25m centres for the down-slope piles.
- Shear modulus stiffness parameters have been applied as follows:
    a. Surface shear modulus = 0kN/m2
    b. Shear modulus gradient = 4000kN/m2 per m
    c. Shear modulus below base = 150,000kN/m2
- Scope of problem = 3, i.e. all 6 degrees of freedom.
- Poisson's ratio for soils and rock = 0.25.
- Free standing length of pile is set at 1.0m (i.e. allowing for a nominal disturbance of ground).
- Pile's Young's Modulus = 2.8e7 kN/m^2 assuming C32/40 grout.
- Piles are modelled as fixed to the pile cap.
- Pile shaft diameter = 225mm, pile base diameter = 196mm.
- The load is applied to a rigid pile cap that is fixed against rotation but able to translate laterally. This is a reasonable model given the fixity of the pile within the pile cap and the relative stiffness between pile and pile cap.

Comparison of Structural frame model results and elastic continuum model

A summary of the pile loads and performance outputs from structural frame model and elastic continuum model are shown below for the up-slope piles.

Table 1. Comparison of structural frame model results and elastic continuum model results

Analysis ref	**Tension pile force**	**Compression pile force**	**Calculated resultant lateral deflection**	**Individual pile bending moment / & shear force**
Slope restoring force: Structural frame calculations	541kN	628kN	10mm	Not available
Slope restoring force: elastic continuum model calculations	531kN	635kN	11.1mm	27kNm / 20kN

The deflections and axial pile loads calculated using Piglet (Elastic continuum model) are consistent with those calculated by the Structural frame model.

Arching check & development of load on to spaced A-frames in response to hillside movement

The discrete micropile "A-frames" were spaced to ensure retention of slope material up-slope of the A-frames. The A-frames rely on soil arching, following minor ground movements, to transfer the slope loads on to them. The spacing of the A-frames is consistent with the recommendations of Carder and Temporal (2000).

There is potential, as a result of minor ground movements around the piles, that significant lateral force will be applied to the piles. Estimates of these forces have been made using published methods and the piles have been designed to resist such forces.

Lateral forces and bending moments could be induced on the piles resulting from downward movement of the hillside. The forces on the A-frame pile caps and micropiles were estimated based on Ito and Matsuio (1975) and Ito et al (1981) as reported in TRL Report 466: *A review of the use of spaced piles to stabilise embankment and cutting slopes*. This method allows the calculation of lateral applied forces per unit length of pile or pile cap. A number of equations and design charts were developed for different soil strengths which enabled the force acting on the pile to be determined. For example the equation for the lateral force (p) acting on a pile per unit thickness of a layer is as follows:

$$p = cA\left(\frac{1}{N_\phi \tan\phi}\left\{\exp\left[\frac{D_1 - D_2}{D_2} N_\phi \tan\phi \tan\left(\frac{\pi}{8}+\frac{\phi}{4}\right)\right] - 2N_\phi^{(1/2)}\tan\phi - 1\right\} + \frac{2\tan\phi + 2N_\phi^{(1/2)} + N_\phi^{-(1/2)}}{N_\phi^{(1/2)}\tan\phi + N_\phi - 1} - \right)$$

$$-c\left(D_1 \frac{2\tan\phi + 2N_\phi^{(1/2)} + N_\phi^{-(1/2)}}{N_\phi^{(1/2)}\tan\phi + N_\phi - 1} - 2D_2 N_\phi^{-(1/2)}\right) + \frac{\gamma z}{N_\phi}\left\{A\exp\left[\frac{D_1 - D_2}{D_2} N_\phi \tan\phi \tan\left(\frac{\pi}{8}+\frac{\phi}{4}\right)\right] - D_2\right\}$$

Slope movement forces on pile caps

The calculations initially consider the forces applied to the pile caps caused by the movement of the surrounding ground. For the up-slope piles, where 600mm wide pile caps are positioned at 2.0m centres the calculated lateral force per metre length (i.e. equal to force on the pile cap for 1m deep piles caps) is 477.4kN. This force is resisted by the A-frame piles and the associated tension and compression leg forces, which can be determined by resolution of forces in the direction of the piles, are 623kN and 400kN respectively. Calculations were also undertaken based on the application of this force on to the Piglet model. The Piglet input parameters are consistent with those stated above other than the applied lateral load of 477.4 x 2 = 954.8kN is used. The resulting outputs from the Piglet calculation are given below.

Table 2.

Peak pile deflections and loads for UP-SLOPE PILES	
Lateral pile group deflection	10.5mm
Compression	422 kN in each vertical pile
Tension	623 kN in each raking pile
Lateral	18 kN in all piles
Individual pile moment	25 kNm in all piles
Pile cap moment	663 kNm

The Piglet results are similar to those calculated using the structural frame model.

The compressive force of 400kN is less than the compression design force of 627kN and is therefore satisfactory. However, the 623kN tension force is greater than the safe working design force of 541kN. The micropile design was therefore based on the greater tension force calculated from this mechanism.

Slope movement forces on piles

The unit pile length slope movement force was determined for 225mm diameter piles at 2.0m centres. The calculated force is 30.9kN per m length of pile. This lateral force can be converted into an equivalent pile bending moment for piles of different ‘free’ lengths. The free length would be that length of the pile that is situated within the moving ground. The bending moments have been determined on the basis that the pile acts as a simply supported vertical beam. This is reasonable given that the pile is fixed at its head (by the pile cap and accompanying pile) and at its toe by embedment into the stable rock. A free length of 1.5m is considered which represent slope slip depths of 2.5m (i.e. 1.5m + pile cap depth of 1m). The following pile bending moments were calculated:

Table 3.

Pile free length (m)	Equivalent pile bending moment (kNm)
1.5	8.7

<u>Structural pile design</u>

The pile’s structural capacity was designed to resist the maximum forces and bending moments from all of the mechanisms considered. The calculated forces from the different mechanism are shown in the table 4 below. The worst case forces and moments are highlighted in bold.

Table 4. Summary of peak pile forces and moments to be resisted (based on up-slope pile frames only which are the most critical case).

Analysis mechanism	**Tension pile force**	**Compression pile force**	**Individual pile bending moment / and shear force**	**Pile cap moment**
Slope restoring force: Structural frame calculations	541kN	628kN	-	-
Slope restoring force: elastic continuum model calculations	531kN	**635kN**	**27kNm** / 20kN	**784kNm**
Slope movement induced forces on pile caps: elastic continuum model calculations	**623kN**	422kN	25kNm / 18kN	663kNm
Slope movement induced forces on piles	-	-	8.7kNm / **23kN**	-

Moment capacity of piles

A circular hollow section (CHS) was included within the upper section of the pile to provide enhanced resistance to bending and shear. The selected section was a 168.3mm outside diameter with 10mm wall thickness. A 40mm (950/1050) full length central bar was installed within the vertical piles and a 50mm (500/600) full length central bar will be installed within the raking piles.

The CHS was designed to be installed in short sections to allow it to be handed on site without the need for lifting gear. The short sections were connected together by virtue of threads cut within the ends of each length. The moment capacity of the CHS at this threaded joint was therefore reduced from the continuous section value calculated above. The reduced moment capacity, allowing for the threads, was determined by considering the same circular section but with half the wall thickness (i.e. 5mm).

Axial capacity of section

Checks on the axial capacity of both piles were carried out to ensure the piles can safely support the designed tension and compression loads. These checks demonstrated that the proposed piles were suitable. The bar diameter of the tension pile was increased to control crack widths within the micropiles.

Pile cap

A pile cap design was undertaken to ensure that the piles are adequately anchored into the pile cap and that the piles cap can resist the calculated moments. The resulting pile cap detail is shown in Figure 11.

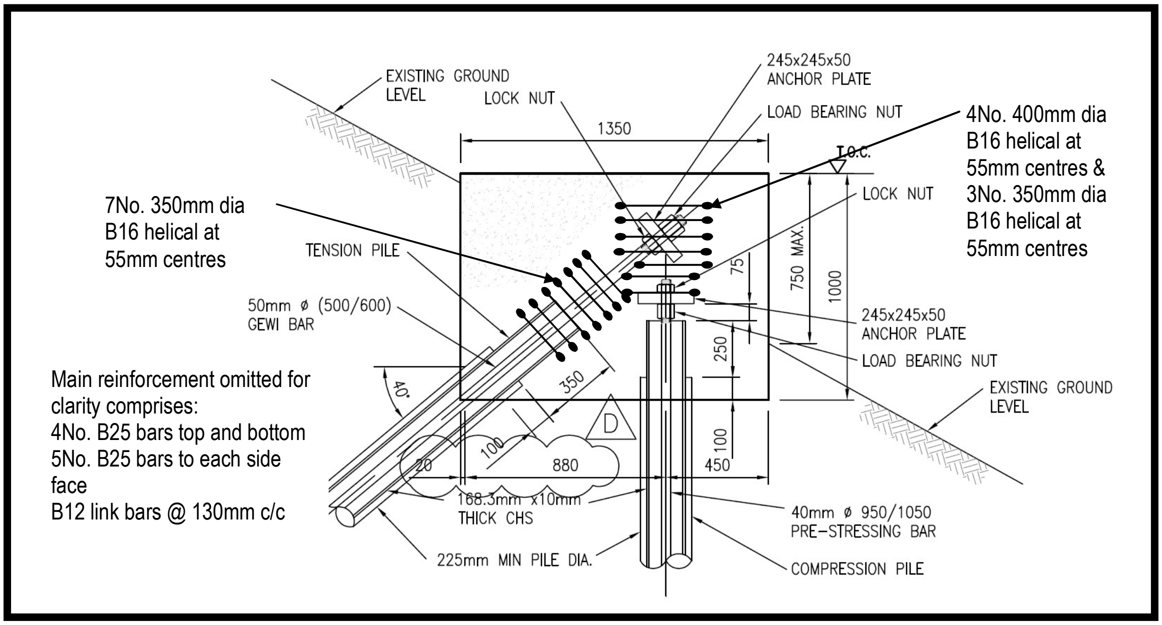

Figure 11. Pile cap detail

Rock socket length calculation

The intact rock is described as moderately strong volcanic Tuff. Rock sockets are to be drilled using open hole rotary techniques and an ultimate rock shaft capacity of 500kPa was considered suitable in such rock. Calculations determined the rock socket length to support the maximum compression load of 627kN (6.5m rock socket required) and the maximum tension load of 623kN (6.5m rock socket required).

LAYOUT OF STABILISATION WORKS & EFFECTIVENESS

The resulting layout of the stabilisation works is shown in Figure 12 below.
Figure 13 shows the results from the ten strain gauges installed within the conduit. A number of the strain gauges recorded progressive movement following their installation. However, it is clear that the rate of increase in strain reduced as the stabilising works were installed. Key construction milestones are labelled on the figure and it can be seen that the strain level across existing cracks essentially halted following the installation of the down slope piles, construction of the associated pile caps and completion of permeation grouting between the conduit and these caps.

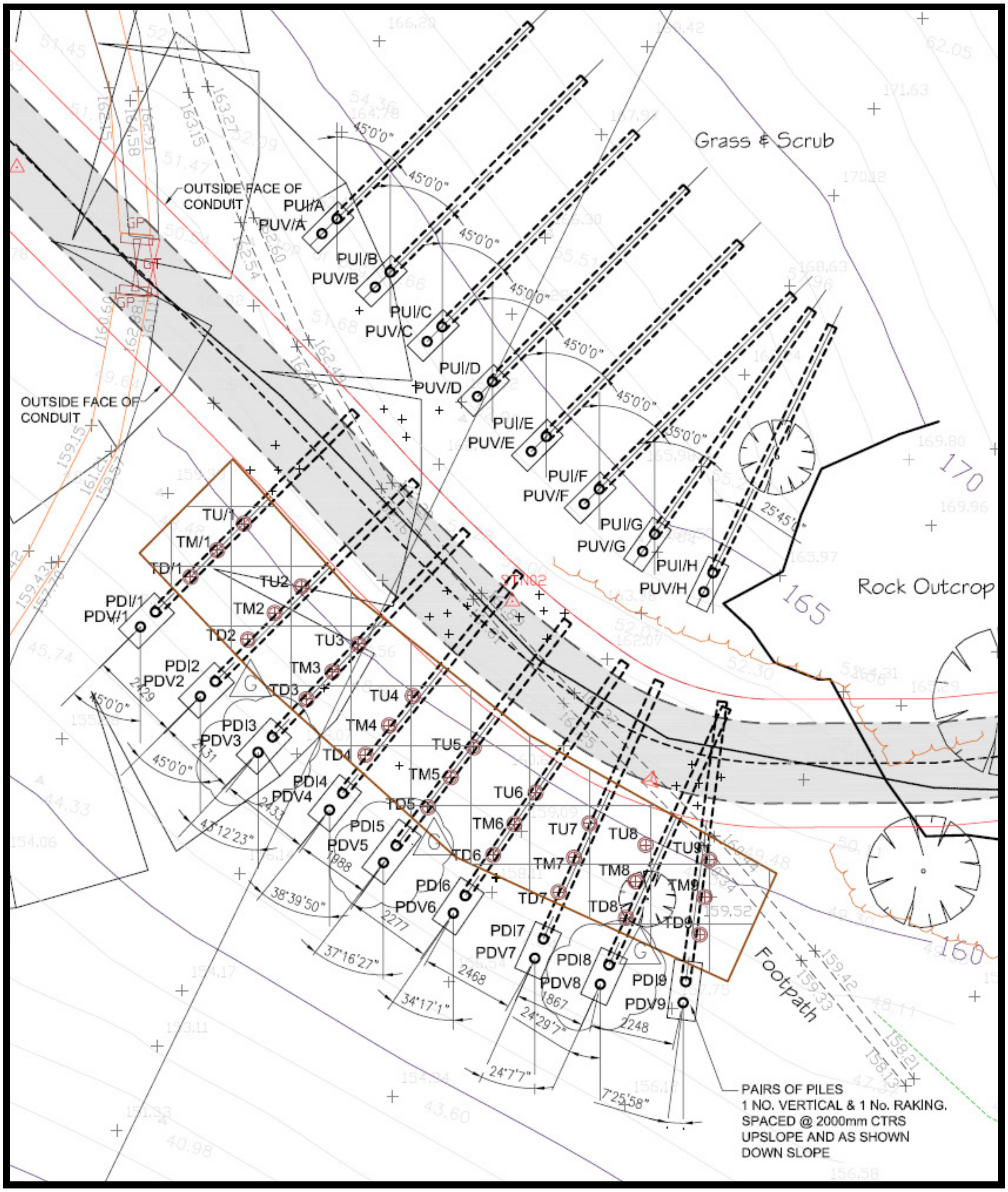

Figure 12. Stabilisation works layout

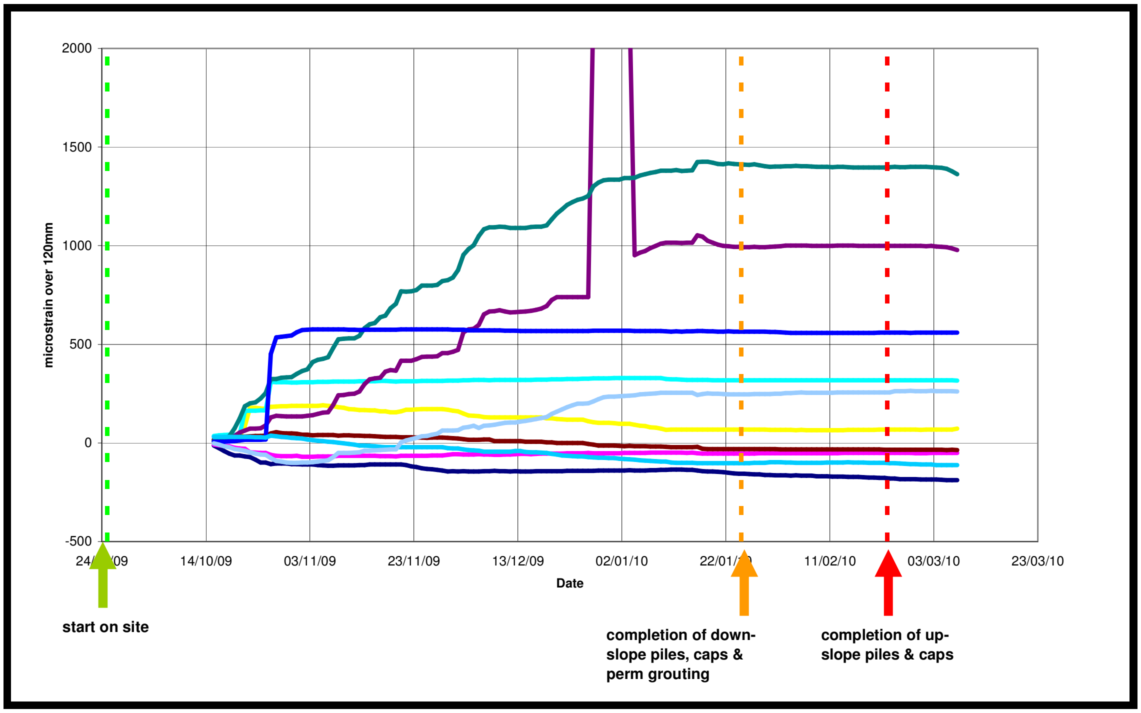

Figure 13: Strain gauge results, measured inside the conduit, across existing cracks.

CONCLUSIONS

A vital section of water infrastructure has been stabilised. The main aspects of the design of the stabilisation works have been set out above together with early indications of the successfulness of the works. The works were undertaken in a challenging environment and under significant restrictions. Strict controls were employed on site to ensure that the conduit was not damage nor the hillside de-stabilised at any point during the works.

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