Technical paper: Validation of electrokinetic stabilisation of M5 Junction 7

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1.0 Introduction

A common feature of conventional methods used to repair failed or failing slopes is that they are based upon the mechanical properties of the materials forming the slope.

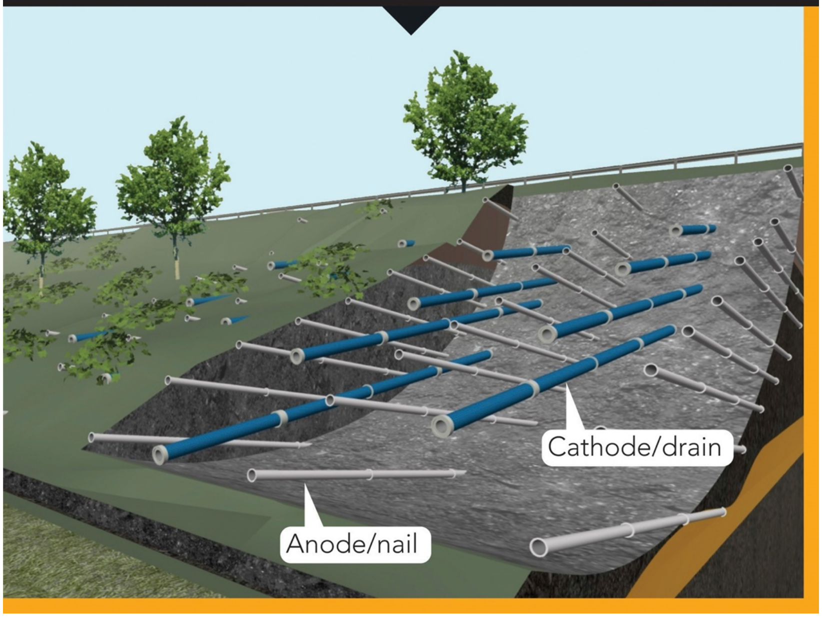
The use of the electrokinetic properties of the soil are usually neglected although BS 8006 (BSI 2011) teaches that the use of electrical energy is an accepted method to increase the shear strength of weak materials in-situ to render them acceptable for construction such as being suitable for the use with soil nails.

Using this approach, the stabilisation of failed or failing slopes can be achieved without the need to remove embankment material and without the need for traffic restrictions/lane closures. This in turn can lead to significant economic and environmental savings, which have been shown to meet the requirements of Construction 2015, (BIS 2013).

The ability to harness the electrokinetic properties of soil economically has been made possible by forming the required electrodes in the form of dual function geosynthetic soil nails (anodes) and geosynthetic drains (cathodes), which are used to provide a short low voltage computer-controlled period of electrokinetic treatment (EK).

Following EK the anodes are retained as permanent soil nails with enhance soil/nail bond and the cathodes are retained as permanent drains. A typical arrangement of electrodes as anode/nails and cathode/drains in a failure plane is illustrated in figure 1.

FIGURE 1: TYPICAL ARRANGEMENT OF ELECTRODES: ANODE/NAILS AND CATHODE/DRAINS AROUND A FAILURE PLANE



EK slope stabilisation is a multifunctional system comprised of four components; dewatering by electro-osmosis; reinforcement; drainage; and soil modification. At the start of treatment an applied voltage gradient across the electrodes installed in the slope results in electroosmotic flow from the anode to the cathode. By draining the cathode and preventing water ingress at the anode, there is a major drop in pore-water pressure which causes an immediate increase in effective stress.

The water removed by electro-osmosis is both interstitial water and vicinal water. Interstitial water is controlled by capillary forces. Vicinal water consists of water molecules layered on the clay particles which can be removed by electro-osmosis. Returning this water to the clay can only be achieved by electro-osmosis using polarity reversal.

During electro-osmosis ion migration from the anode occurs, which can be enhanced or controlled by selecting the structure of the anode together with the optional use of conditioning fluids. Cementing round the anode increases the anode/soil bond referred to as the interface shear strength (IFS). The cemented zone also provides improved frictional characteristics in the soil. The enhanced bond increases the pull-out resistance of the anode/soil nail and is permanent, (Milligan, 1995).

EK treatment of soil has beneficial effects in changing the chemical nature of the soil, including cementation, precipitation, cation exchange, and (in some soils) changes in particle size distribution. The physio-chemical effects usually act together with consolidation (where it occurs) to increase the bulk soil shear strength parameters, soil stiffness and reduce plasticity, (Pugh 2002).

Electrokinetic geosynthetic materials (EKG) were developed with EPSRC and TSB funding supported by industry, (Jones et al, 2017).

EK strengthening of slopes and the economic and environmental benefits which the system offers have been described by (Lamont-Black et al, 2012; Jones et al, 2014 and Lamont-Black et al, 2016).

EK treatment of slopes requires energy to complete, which includes monitoring of the process to ensure that the required improvement of the properties of the failing slope has been achieved. This paper describes the verification methods, which are used during the EK treatment augmented by a number of post treatment investigations.

2.0 M5 J7 case history

The verification tests relate to the EK stabilisation of an unstable embankment, which formed part of junction 7 on the M5.

Progressive shallow failures of the embankment had been observed since 1999.

Repair of failures on the western side of the embankment using granular replacement and slope drainage were successfully carried out in 2000. At the same time a planting scheme of deep rooting trees was implemented on the eastern side of the embankment to improve stability, but this proved ineffective with continuous movement, which threatened the overall stability of the southbound slip road.

The original remediation scheme was for a soil nail solution, which would require the removal of the embankment material associated with the slip together with extensive traffic disruption. Value engineering indicated that a solution based on EK could be achieved without the need to remove any fill, would provide savings in cost and carbon footprint as well as eliminate the need for lane closures on the M5 interchange.

Details of the design and construction of the EK remediation were presented at the 16th European Conference on Soil Mechanics and Geotechnical Engineering, (Jackson et al, 2015). The length of embankment treated was 265m, varying between 6m and 10.5m in height and with a slope angle of 26° to 29° (figure 2).

2.1 Preliminary engineering assessment

A preliminary assessment of the stability of the slope was carried out to mimic the failure mechanism apparent on site and to determine the likely strength of the slope materials at failure. For simplicity it was considered that the soil related to a Mohr-Coulomb failure criterion ($\tau = c' + \sigma' \tan\varphi'$), where the shear stress, τ depends on the effective stress acting on the shear plane. The stability of the embankment was analysed using Slope-12R (Geosolve, 2003).

The stability analyses used to assess the geotechnical parameters at failure were performed by reducing the peak shear strength parameters of the embankment fill assuming a typical value of pore water pressure ratio, r_u, of 0.1-0.2. The analyses performed on representative sections of the embankment showed a failure mechanism in the top 3m of the slope consistent with the appearance of the failed embankment when the shear strength parameters of the embankment fill were $\varphi' = 20°$ and $c' = 0 – 1.5$kPa. In addition, the analyses identified a potential deep-seated rotational failure.

3.0 Initial ground investigation

An initial ground investigation of the failing embankment was carried out in January 2010 consisting of 11 window sample boreholes, six rotary boreholes, 17 in-situ CPT tests followed by laboratory testing, which included classification, strength and chemical tests, (Amey, 2011).

The ground investigation identified the embankment core as reworked firm to stiff weathered Charmouth Mudstone (Lower Lias Clay), which is a highly plastic clay with problematic shrink-swell characteristics, overlain by a mantle of variable soft to firm glacial or terrace deposits identified as reworked Mercia Mudstone (table 1). The underlying geology was the Mercia Mudstone Group (figure 2).

FIGURE 2

(a) View of M5 J7 embankment during Electrokenetic treatment

(b) Typical cross section of the slope based on the interpretation of Ground Investigation

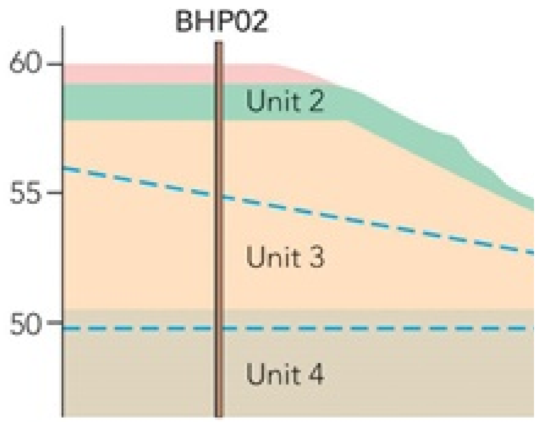


TABLE 1: SUMMARY OF THE GEOLOGICAL UNITS ON THE M5 JUNCTION 7

UNIT ID	STRATA	DESCRIPTION	APPROX. THICKNESS
Unit-1	Road materials	Tarmacadam and Granular Made Ground	<1m
Unit-2	Reworked Mercia Mudstone	Soft to firm reddish brown slightly sandy slightly gravelly clay with occasional rootlets. Gravel is fine to medium, sub angular to sub rounded consisting of mudstone	0.5 – 2.8m (average 1.4m)[1]
Unit-3	Reworked Lias Clay (Charmouth Mudstone)	Firm to stiff and very stiff, occasionally soft, grey to dark grey occasionally greenish grey/brown, blue grey and reddish brown, mottled yellow slightly sandy and sandy slightly gravelly and gravelly CLAY	3.6-8.5m (average 6.2m)[1]
Unit-4	Mudstone Group (Natural ground)	Stiff to Very stiff fissured friable reddish brown locally mottled bluish grey slightly sandy slightly gravelly CLAY	Up to 12m proved

The groundwater levels were below the motorway main carriageway within the underlying Mercia Mudstone Group, although some groundwater strikes had been recorded within the embankment. Four areas of failure along the embankment were observed resulting generally from the development of a failure surface in the base of the upper softened zone of embankment fill at the interface with the re-worked Lias Clay embankment core material.

4.0 EK material properties

In addition to the ground investigation, laboratory tests were undertaken to determine the EK properties of the embankment materials. The properties used in the remedial design were; the electrical conductivity, coefficient of electro-osmotic permeability, electro-osmotic consolidation and post-electrokinetic treatment shear strength. These determine the effects of EK treatment, the required voltage gradient and duration of the active treatment period required to stabilize the embankment.

Electrical conductivity was determined according to BS1377, 1990 Part 3, while both the coefficient of electro-osmotic permeability and electro-osmotic consolidation characteristics were determined using a modified Rowe cell, in which a voltage gradient is used in place of a hydraulic gradient, (BS1377, Part 6, figure 1d; Hamir et al, 2001 and Pugh, 2002).

Characterisation of the anode (reinforcement) action was determined using a large cell apparatus, which simulates the EKG treatment and characterised the strength improvements of the anode-proximal soil and the bond strength (adhesion) developed between the anode and the soil material.

The EK laboratory tests provided data on changes of moisture content, shear strength and plasticity of the soil as well as the recession of anode material which occurs during the active treatment period. In addition, shear box tests were undertaken to assess the effect of electro-osmotic consolidation of the material associated with the shear plane, which was at residual strength.

The test comprised the consolidation of highly sheared clay surface produced by repeated shearing in the shear box. This was then subjected to consolidation at an applied load representative of the additional load expected as a result of electro-osmotic treatment.

The consolidated material was then unloaded to simulate removal of the voltage gradient and subsequent dissipation of the pore water suction and the shear strength tested again. The consolidated unloaded shear box test indicated an improvement in the residual values of cohesion for unit-2 of 3KPa; and the angle of shearing resistance increased from 14° to 18°.

The consolidated drained triaxial tests showed similarly improved values of c' and φ' after electroosmotic treatment compared to the characteristic values determined in the preliminary engineering assessment. A summary of the EK tests and the characteristic values are given in table 2.

TABLE 2: ELECTRO-OSMOTIC MATERIAL PARAMETERS FOR UNIT-2 AND UNIT-3

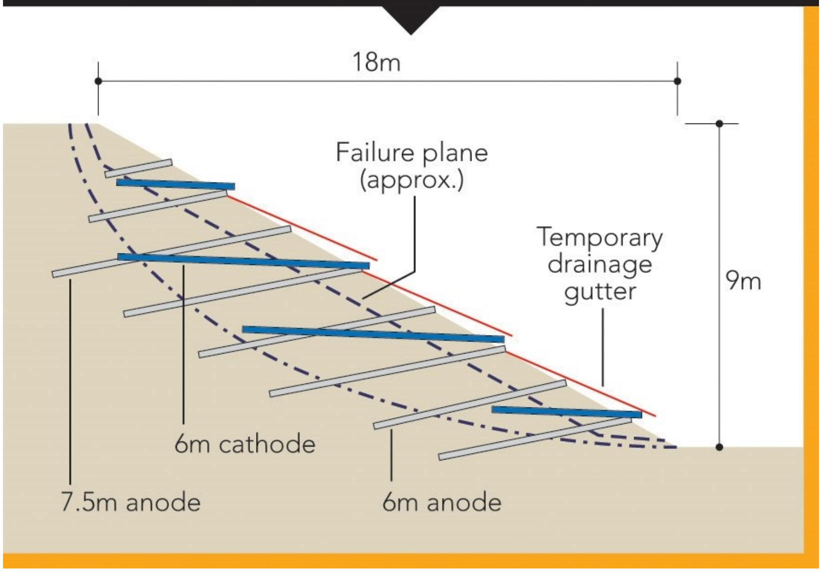
ELECTRO-OSMOTIC TESTING AND PROPERTIES	GEOLOGICAL UNIT	
	UNIT-2: REWORKED MERCIA MUDSTONE	UNIT-3: REWORKED LIAS CLAY
Electrical conductivity (mS/m)	46.7	46.7
Coefficient of electro-osmotic permeability (m^2/sV)	4×10^{-9}	3×10^{-9}
Electro-osmotic consolidation (MN/m^2)	Reduction in void ratio by 14%	
Length of electrokinetic treatment needed to produce 90% consolidation	6 weeks	
Change in pressure due to EO - $\Delta\sigma'$ (kPa)	57	70
Coefficient of volume compressibility - m_v (m^2/MN)	0.18	0.17
Anode pull-out resistance interface shear strength, IFS and Adhesion, A (large cell test)	IFS=88.71 kPa A=7.93 kN/m	-
Shear strengths determined from back analysis of the failure mechanism	$c'=0-1.5$ $\phi'=20°$	$c'=0$ $\phi'=18°$
Consolidated drained strength - triaxial tests	$c'=16$ kPa $\phi'=20°$	$c'=18$ kPa $\phi'=24°$
Residual strength - shear box	$c'=0$ kPa $\phi'=14°$	-
Consolidated unloaded residual strength - shear box	$c'=3$ kPa $\phi'=18°$	-
Anode surface recession	0.64mm	

5.0 Remedial design

Remediation involves the installation of an array of electrodes (anodes and cathodes) into the failed slope to an adequate depth below any potential failure, followed by a period of EK treatment to increase the shear strength of the slope material as detailed in BS 8006 (BSI 2011).

Investigation of the embankment failure mechanism showed that stability could be achieved by effecting an improvement to the shear strength of the materials forming the embankment to a depth of 3m. To cover the potential deep-seated failure mechanism an EK treatment depth (anode length) of 7.5m was required with the cathode (drains) being 6m (figure 3).

FIGURE 3: TYPICAL CROSS SECTION OF ELECTRODES USED IN THE MODEL



The electrodes used to stabilise the slope were installed in columns arranged perpendicular to the slope contours at spacings varying between 1.5m and 2m. A closer spacing of columns equates to a higher voltage gradient (with concomitantly higher treatment intensity) and a higher density of reinforcing and drainage elements. The closer spaced columns were installed in sections of the slope assessed to have the lowest stability. A total of 612 anodes and 308 cathodes were used in the design.

Following installation, the electrodes were wired to a DC rectifier transformer powered by a diesel generator and a voltage (80 – 100V) was applied over a treatment period of six to eight weeks. The variation in treatment period was to account for local variations in electrical conductivity. At the end of the treatment the anodes were transformed into soil nails with a design life of 100 years by the addition of steel tendons grouted into the anode tubes. The cathodes were retained to act as permanent horizontal drains.

6.0 Verification of EK treatment

During the EK treatment phase a number of tests are performed to monitor and verify that the treatment has achieved the objective of stabilising the failed/failing slope. These tests include monitoring the electrical charge applied to the electrodes, recording the volume of water discharged from the cathodes and load testing a selection of the anode/nails post EK treatment.

The crest of the embankment was surveyed to monitor embankment movement throughout the EK treatment. In total, 28 observation points were set up at 10m intervals at the top of the embankment and monitoring carried out between 20 April and 2 June 2012.

As the M5 scheme was one of the first remedial schemes to adopt the EK technique the usual tests were augmented by a number of post construction verification tests carried out with the following objectives:

- To develop a better understanding of how the technique translates from theory to practice and from laboratory to field
- To confirm that the technique delivers what it is designed to do
- To identify factors critical to the successful implementation of the technique

6.1 Electric charge (anodes)

The interface total discharge density is the total or charge passed per unit area across the electrode/soil interface during the EK treatment period. This variable determines the development of the adhesive component of the anode-soil bond. At the end of the six-week EK treatment, the average anode interface total discharge density for the whole array was 10.1MC/m^2 with the average electrical energy dissipated being 22.1MJ/m^3.

The predicted charge based upon the laboratory testing required to develop the required bond with the anode nails was 10.7MC/m^2 and the energy to develop 90% consolidation was 15MJ/m^3. Thus, within acceptable limits, the electrode array behaved in a similar manner to the laboratory tests upon which the design was based.

6.2 Water discharge (from cathodes)

Average cathode discharge for the array using drip meters was 20 litres per day. However, it was apparent that much of the discharge was not collected and the total volume of water discharged during treatment was not established. After the end of the EK treatment phase the water discharge ceased. Figure 4 shows that the EK treatment coincided with a period of intense rainfall, however, this had no effect on the treatment.

6.3 Embankment crest settlement

EK treatment causes electro-osmotic consolidation, which in turn causes settlement of the embankment. This is a maximum at the anode and zero at the cathode. Total settlement, δ, for the treatment zone can be estimated using the expression:

$$\delta = \Sigma m_v \Delta\sigma' h \qquad \text{eq 1}$$

Where m_v is the coefficient of volume compressibility corresponding to a change in effective pressure, $\Delta\sigma'$, due to electroosmotic consolidation and *h* is the average thickness of the treated zone (1.75m). The laboratory testing showed that the coefficient m_v ranged from (0.18 and 0.17m^2/MN) corresponding to an effective stress $\Delta\sigma'$ of 57 and 70KPa for reworked Mercia Mudstone and reworked Lias Clay, respectively (table 2). The average thickness, *h*, of the treatment zone, extended from 0.5m to 3m depth. Thus, the maximum settlement at the anode was estimated at 28mm. As, the anodes were installed at spacing of 3m to 4m along the embankment the predicted settlement was 28/(1.5 or 2) = 14mm or 19mm. This agreed with the measurement of levels at the crest of the embankment, where the maximum settlement recorded was 16mm.

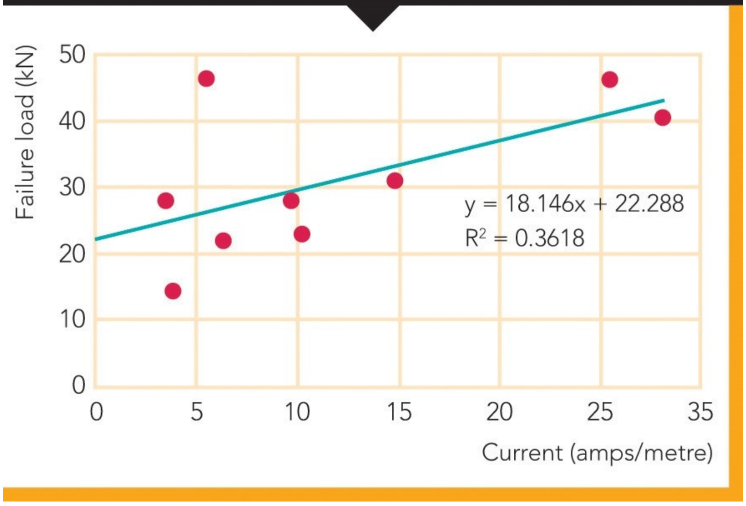
6.4 Load testing of anodes

Following the electrokinetic treatment phase proof load testing of a number of selected anodes from each treated section was used to provide direct confirmation that the required soil/nail bond had been achieved (Jones, et al., 2016). The load testing carried out showed variable results with three out of nine results below the required working load of 24kN (under serviceability conditions). The correlation between pull-out load and (electric) current/charge dissipated for each tested anode were normalised with respect to the respective treatment section (table 3 and figure 5).

TABLE 3: PULL-OUT TEST RESULTS AND (ELECTRIC) CURRENT DATA (WITH AND WITHOUT NORMALISATION)

NAIL ID	EMBANKMENT SECTION	FAILURE LOAD, Pu	ANODE AS PROPORTION OF SECTION	AVERAGE CURRENT TREATMENT SECTION	AVERAGE CURRENT FOR ANODE	AVERAGE CURRENT/m LENGTH
		(KN)	(%)	(AMPS)	(AMPS)	(AMP/m)
69E	1	22	1.03	184.32	1.90	0.25
40C	3	23	1.25	247.80	3.10	0.41
21F	5	31	1.73	258.72	4.46	0.60
62E	2	28	0.38	282.72	1.07	0.14
55E	2	28	1.24	282.72	3.51	0.39
48D	3	14.5	0.56	247.80	1.39	0.15
35E	4	46	0.93	181.44	1.69	0.23
24C	5	46	2.95	258.72	7.63	1.02
13F	6	40.3	2.95	285.18	8.43	1.12

FIGURE 5: PULL-OUT FAILURE LOAD V ELECTRIC CURRENT FOR ALL TESTS CONSIDERING AVERAGE FOR TREATMENT



As shown in figure 5 there is a good correlation between the average current per treatment panel and ultimate pull-out resistance developed, such that the lower results are generally at the northern end of the site which comprised drier soil in the shorter shallower slopes. The linear correlation shown in Figure 5 can be expressed as:

$$P_u = aC_T + b \qquad \text{eq 2}$$

Where P_u is the ultimate pull-out force (kN), C_T is the average currents per treatment panel (amps/m) and *a* and *b* are coefficients related to the electrodes design arrangements and electroosmotic properties of the embankment materials. At the M5 embankment *a*=18.14 and *b*=22.28.

Some variation of the pull-out tests results may be attributed to installation issues. An oversized anode drive head was used to install the anode electrodes, which may have limited effective contact between the anode and the stiffer embankment materials. This did not inhibit the dewatering, consolidation and mass strength gain, but did appear to have affected the development of the enhanced cemented bond between the anode and the soil in some locations.

6.5 Post-EKG treatment ground investigations

After completion of the EK treatment, seven exploratory holes were drilled in November 2012 to provide soil samples to determine the change in soil properties. A further ground investigation was undertaken in April 2014 18 months after the completion of the remediation works. This comprised of two window sample boreholes and six in-situ CPT tests. Samples were taken at two locations close to previous investigation holes to provide as direct comparison as possible.

Tests results from the post-treatment ground investigations were analysed and compared with the results obtained from the initial (pre-treatment) ground investigation. In addition to the CPT testing, the results obtained from tests covering: natural moisture content, soil consistency using Atterberg Limits, void ratio and degree of saturation, were considered in the verification study.

7.0 Results and discussion

7.1 Moisture content and plasticity

A total of 50 natural moisture content and liquid and plastic limits tests were carried out on samples from the exploratory boreholes at various depths. The results show a general reduction in the moisture contents and liquid and plastic limits in both the reworked Mercia Mudstone and Lias Clay, following EK treatment (figures 6 and 7).

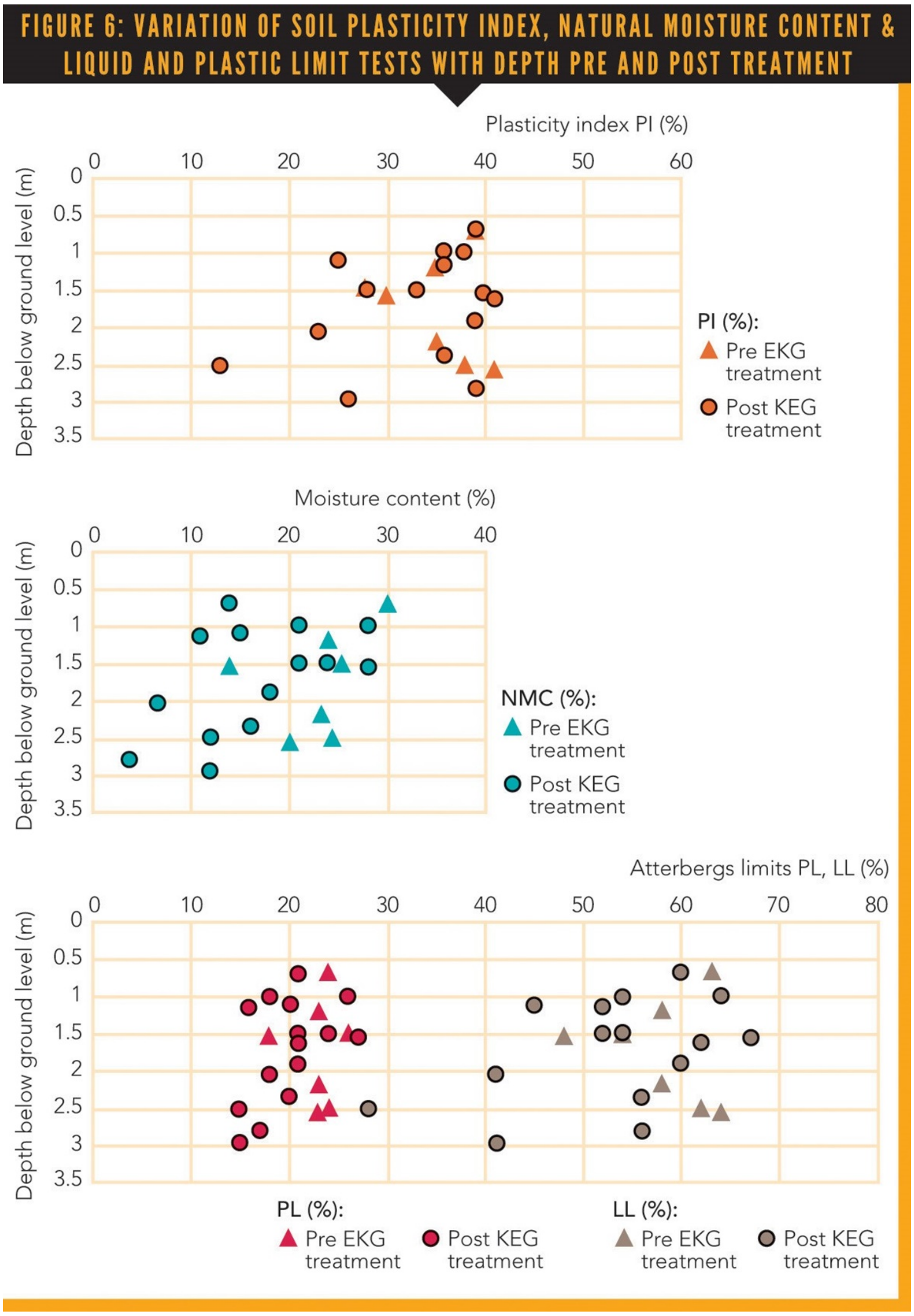


FIGURE 7: PLOT OF MOISTURE CONTENT IN RELATION TO PLASTIC LIMIT FOR THE TREATED EMBANKMENT MATERIAL FROM 0.5 TO 3M BELOW SLOPE SURFACE

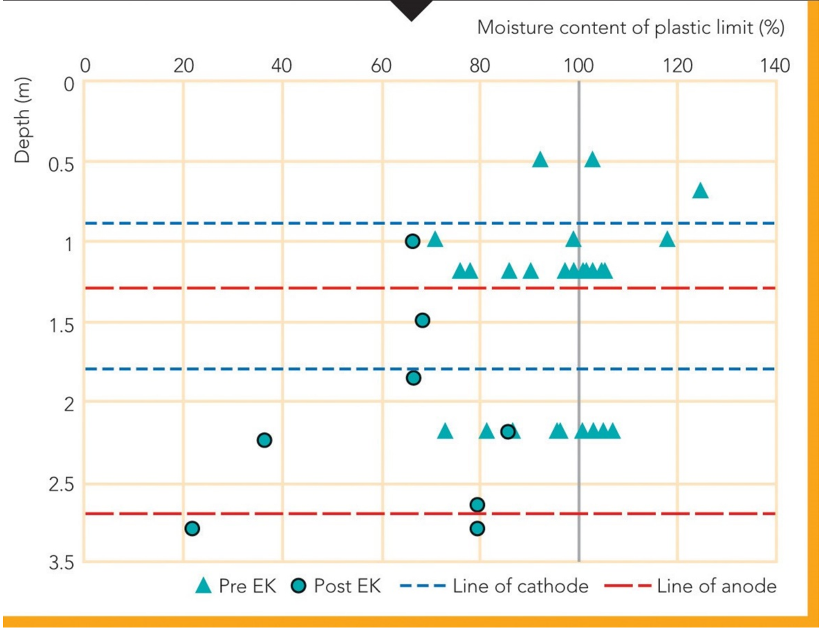


Figure 7 shows the pre-treatment moisture contents lay at or around the corresponding plastic limits, whereas the post-treatment data indicates that the EK treatment reduced the moisture content well below the plastic limit. The reduction in soil plasticity is associated with physio-chemical changes in the soil including cementation, precipitation, and flocculation effects which act together with consolidation and shear strength improvement, (Milligan, 1994; Pugh 2002). Statistical analysis carried out on the data shown in figures 6 and 7 is shown in table 4. The reduction in natural moisture confirms the discharge of water observed during the active treatment period (figure 4).

TABLE 4: CHANGE IN AVERAGE VALUES (ARITHMETIC MEAN) OF MOISTURE CONTENT, PLASTICITY LIQUID LIMITS, AND PLASTICITY INDEX FOR THE EMBANKMENT MATERIAL*

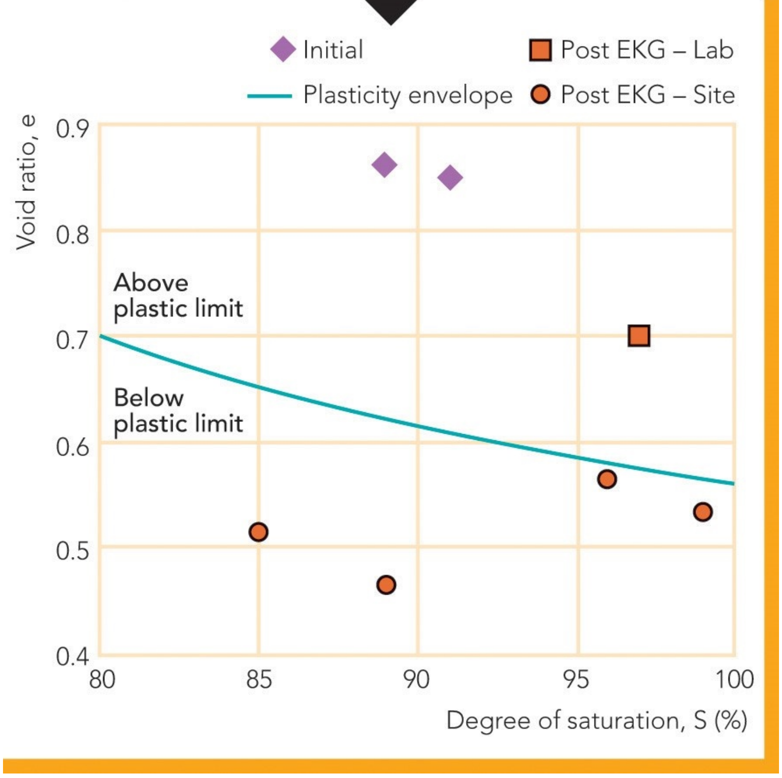
	NMC (%)	LL (%)	PL (%)	PI (%)
Pre-treatment	23.0	58.1	23.0	35.1
Post-treatment	16.3	52.8	20.0	32.8
Change (%)	-29.1	-9.2	-13.0	-6.7

From 0.5* to 3m bgl (depth to which treatment took place). Any data above 0.5m depths was omitted, as it is likely that the near surface material will have been heavily affected by weather conditions.
During the EK treatment phase the site was subjected to heavy rain, Figure 4

7.2 Void ratio and degree of saturation

Figure 8 presents the degree of saturation against void ratio for pre and post EKG treatment from the treated area. By assuming a typical specific gravity of 2.7 and taking the mean of the pre and post-treatment plastic limits (21%), a plasticity envelope is defined for differing values of void ratio and degree of saturation.

FIGURE 8: PLOT OF VOID RATIO V DEGREE OF SATURATION



The post-treatment material lies below the plasticity threshold, whereas the pre-treatment and laboratory treated EKG materials are above. EK treatment produced a marginal increase in the mean value of the degree of saturation, while the mean value of void ratio exhibited a significant decrease of up to 32%.

These results indicate that electro-osmosis removed water from a partially saturated soil, which agrees with previous observations and that the combination of a change in void ratio with little or no change in saturation ratio indicates that the soil response to electro-osmosis was consolidation rather than drawing of air into the void spaces, (Pugh, 2002). The implication of this being that the EK treatment produced a significant reduction in void space.

7.3 Undrained shear strength

The pre-treatment undrained shear strength was derived from SPTs and hand shear vane testing while that from post-treatment was taken solely from hand shear vanes. The range of undrained shear strength values for pre and post treatment testing is shown in table 5.

TABLE 5: UNDRAINED SHEAR STRENGTH VALUES – PRE AND POST-TREATMENT

UNDRAINED STRENGTH, cu (KPA)	PRE-TREATMENT		POST-TREATMENT	
	Unit-2	Unit-3	Unit-2	Unit-3
Minimum	23	26		
Mean	48	41	≥140	40
Maximum	87	72		

In the reworked Mercia Mudstone (unit-2) the undrained shear strength was strengthened from soft to firm to stiff, to firm to stiff to hard. In some locations, surface materials to about 0.5m remain soft to firm, below that the undrained shear strength increased after treatment by an average of 90kPa.

The comparison of the available data for the Reworked Lias Clay (unit-3) indicates that the treatment was less effective at increasing undrained shear strength in this unit, table 5.

7.4 Effective shear strength

The first ground investigation carried out directly following the EK treatment in 2012 included five consolidated undrained single stage and four consolidated undrained multistage triaxial tests with pore water pressure measurement carried out on samples of the treated reworked Mercia Mudstone and reworked Lias Clay.

Based on Mohr–Coulomb failure criteria, the tests results of the triaxial testing were used to draw Mohr circles for each material and then the best common tangent to the circles was used as a strength envelope to determine the effective shear strength parameters (c' and φ').

The results from the 2012 triaxial tests were inconsistent. This was attributed to difficulties with sampling and testing to the required quality using thin walled sampling tubes in stiff materials. The results from the 2014 ground investigation showed improved values of c' = 5KPa and φ = 26° to 27° compared with the results from back analysis of the slope failures which indicated c' = (0 – 1.5) kPa and φ = 20°.

No improvement was apparent in the effective stress parameters of the embankment core material below the targeted treatment zone.

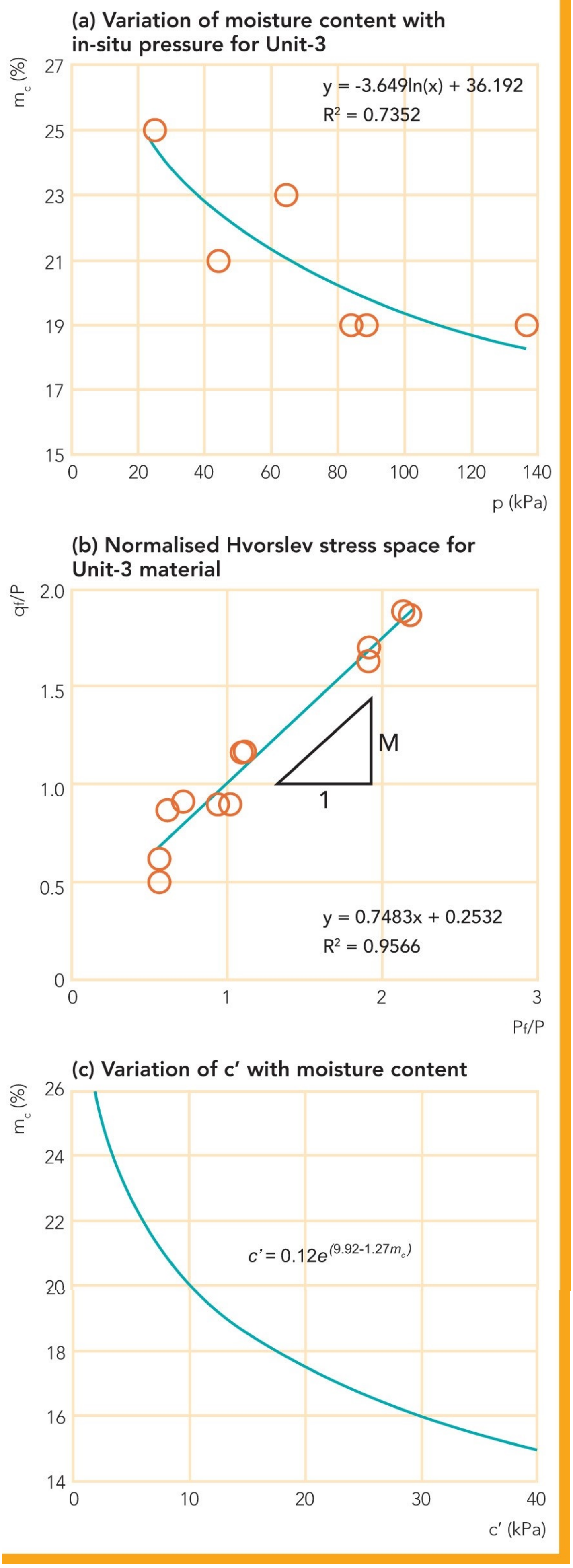
The improved effective shear strength in the treated zone is consistent with the results obtained from the shear strength laboratory tests. The field observations of water discharge as well as the significant decrease of the void ratio imply that electro-osmotic dewatering resulted in consolidation of the failed and soft soil areas, leading to an increase in the overall shear strength.

7.5 Moisture content and effective cohesion

Previous work conducted by Wroth (1985) showed that the effective cohesion c' may be expressed as an exponential function of the moisture content, m_c, and follows Hvorslev failure criterion. This implies that any reduction in water content may result in a significant increase in effective cohesion. The field observation of water discharge during EK treatment implies that the moisture content had been reduced in the embankment materials, consistent with the laboratory testing results of natural moisture content.

To study the correlation between c' and water content m_c, the triaxial test results from the post treatment investigation were investigated following the procedure outlined by Wroth (1985). By examining the relationship between natural moisture content and in-situ pressure and then using the in-situ pressure to normalise values plotted in the Hvorslev stress space it is possible to produce a unique relationship between drained cohesion c' and m_c.

FIGURE 9


(a) Variation of moisture content with in-situ pressure for Unit-3
m_c (%)
27
25
23
21
19
17
15
0
20
40
60
80
100
120
140
p (kPa)
y = -3.649ln(x) + 36.192
R² = 0.7352
(b) Normalised Hvorslev stress space for Unit-3 material
qf/P
2.0
1.5
1.0
0.5
0
0
1
2
3
Pf/P
M
1
y = 0.7483x + 0.2532
R² = 0.9566
(c) Variation of c′ with moisture content
m_c (%)
26
24
22
20
18
16
14
0
10
20
30
40
c′ (kPa)
c′ = 0.12e^(9.92-1.27m_c)

This procedure has been adopted for the reworked Lias Clay (unit-3). The variation of m_c with respect to in-situ pressure, *p*, is given by the equation (figure 9a):

$$m_c = -3.65 ln\ (p) + 36. \qquad \text{eq3}$$

Values of deviatoric stress q_f plotted against mean effective pressure p_f (both normalised to in-situ pressure *p*) in the Hvorslev stress space for the multistage triaxial tests are shown in figure 9b. Thus, the effective stress parameters can be calculated by the following equations:

$$\varphi' = \sin^{-1} \frac{3M}{6+M} \qquad \text{eq4}$$

$$c' = intercept\ \frac{(3 - \sin \varphi')}{(6 \cos \varphi')} \qquad \text{eq5}$$

This gives a value of φ' = 20° and a relationship of c' with in-situ pressure of:

$$c' = 0.12p \qquad \text{eq6}$$

By substituting equation 6 into equation 3 and rearranging, a unique relationship of drained cohesion with respect to moisture content can be derived (Figure 9c):

$$c' = 0.12e^{(9.92-1.27m_c)} \qquad \text{eq7}$$

By applying equation 7 to the values of moisture content from the triaxial tests it is possible to determine a range of drained cohesion for the Unit-3 material. This provides a variation in c' of 5.9kPa to 30.3kPa (Table 6). As two of the tests lie outside the treatment zone of 0.5m bgl to 3m bgl (BHA 16UT and BHB 14UT) a revised range of c' varying from 5.9KPa to 10.1KPa can be obtained for unit-3 after EK treatment.

TABLE 6: COMPARISON OF CALCULATED VALUES TO MEASURED VALUES OF c' FOR TRIAXIAL TESTS

BOREHOLE	SAMPLE	DEPTH m BGL	MC (%)	CALCULATED c' (KPA)	CALCULATED φ'[2]	MEASURED c' (KPA)	MEASURED φ'
BHA	14UT	3.00	20	10.1	20	12.5	17.1
BHA[1]	16UT	3.45	20	10.1	20	6.4	19.2
BHB	8UT	1.80	22	5.9	20	7.4	17.5
BHB[1]	14UT	3.40	16	30.3	20	1.3	21.3

[1] Depth of samples is outside of the effective treatment zone of 0.5mbgl to 3mbgl
[2] Calculated from Figure 9(b)

When compared with the results obtained from back analysis of the slope failures (φ = 20° and c' < 1.5kPa), Table 2, these results indicates a significant improvement in c' in unit-3. As c' plays a critical role in the long-term stability of a slope, it can be concluded that the stability of the treated slope has improved.

7.6 Slope stability analysis

The stability of the embankment incorporating the improved shear strength of the soil materials and the addition of the anodes acting as soil nails was assessed to determine how the factor of safety (FoS) had been improved by the EK treatment.

As some of the pull-out results were below the required design working load, slope stability analyses were conducted to re-assess whether the reinforcement, provided by anodes as soil nails, could provide sufficient resistance to ensure the required factor of safety.

Values of the adhesion developed between the nails and the soil were derived from back analyses of the pull-out tests. The pull-out values ranged from 14.5KN to 46KN, which provided a variation in soil/nail adhesion of 7KPa to 50KPa.

For the mean value of pull-out resistance of 31KN the soil/nail adhesion is 25KPa, which was adopted in the slope stability analyses. The analyses also adopted the improved shear strength parameters within the treatment area extending from 0.5m to 3m below slope surface.

The stability of the embankment was analysed using Slope-12R (Geosolve, 2003). Figures 10 and 11 show the analytical results of a typical section of the embankment using drained and undrained soil parameters respectively, where a minimum FoS above the threshold value of 1.3 required by HA68/94 (Highways Agency 1994), were obtained indicating that the EK treatment had resulted in stabilising the failing embankment.

FIGURE 10: RESULTS OF SLOPE STABILITY ANALYSIS USING EFFECTIVE (DRAINED) SOIL SHEAR STRENGTH PARAMETERS AFTER TREATMENT

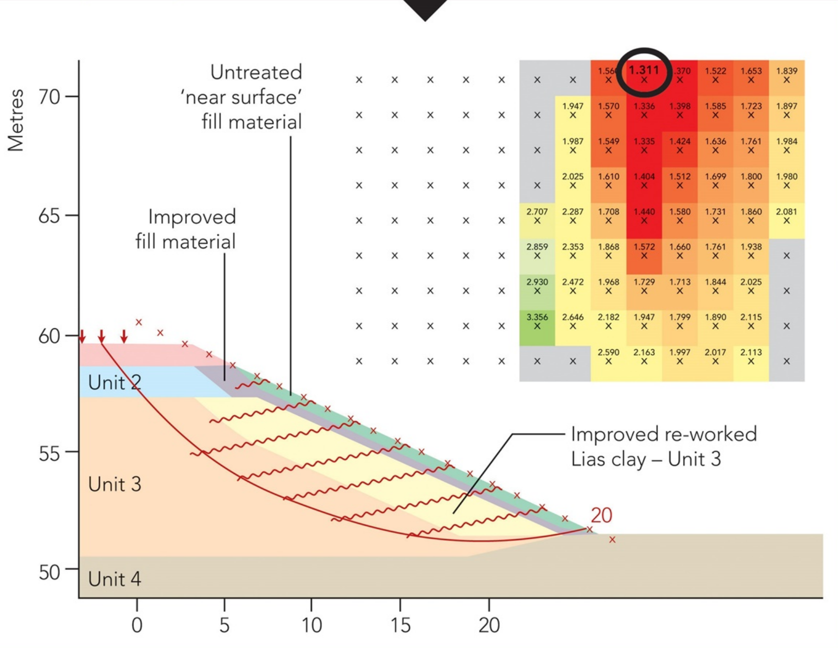
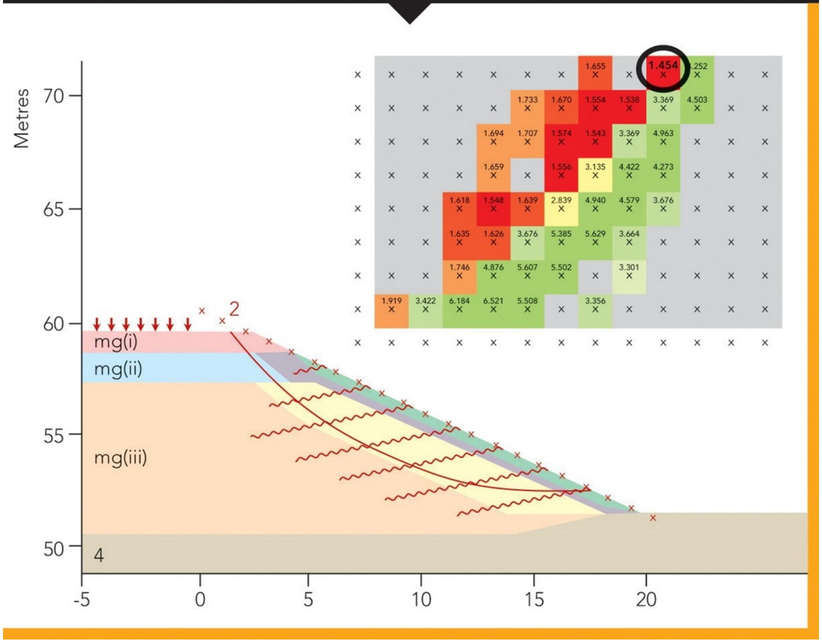


FIGURE 11: RESULTS OF SLOPE STABILITY ANALYSIS USING TOTAL (UNDRAINED) SHEAR STRENGTH PARAMETERS AFTER TREATMENT



8.0 Conclusions and recommendations

Verification testing of the M5 Junction 7 embankment treated by EK remediation in 2012 and 2014 showed a significant reduction in moisture content and improvement in drained and undrained shear strength properties within the targeted treatment zone, which indicated that the treatment had brought a satisfactory improvement to slope stability. Improvement was also observed with some change in moisture content and undrained shear strength in the embankment core materials, but improvement in drained shear parameters was not proved below the targeted treatment zone.

Pull-out test results identified implementation and installation issues with some anodes that appear to have inhibited full development of the anode-soil bond. However, resilience in the design provided an adequate factor of safety. The use of an oversized anode head may have limited effective contact between the anode and the soils. Thus, following the lessons learned from the M5 scheme, the design now used in practice comprises a drive cone, which is flush with the anode body. No anode nails have since recorded bond strengths lower than the proof load. By considering the charge passed during the EK treatment with respect to the area of each anode it is possible to derive a value, which can be used for validation of anode pull-out resistance in the field.

At the M5 J7 embankment the electrodes (anodes and cathodes) were installed on straight lines on a curved embankment. Further improvement might be achieved if the layout of anode and cathode lines are designed to match the topography of the embankment i.e. on concave/convex lines not on straight lines.

A high level of pH was identified in the discharged water from the cathodes, and this elevated alkaline content could have health and safety and environmental implications, therefore suitable safety measures should be adopted when dealing with this water. Subsequent projects have ensured that discharge is collected and then tested before being released.

The design analysis was based on the assumption of a uniform electric field, which may not be ideal for representing the in-situ condition due to complexity of electrode configuration, variable boundary conditions and anisotropic soil properties. Improvement in the analytical model could achieve more economical designs.

The use of the EK strengthening system has been shown to provide cost saving, environmental benefits and to eliminate the need for traffic management when compared with other treatment methods (table 7). These savings demonstrate that EK treatment option has the potential for widespread application for remediation of failed or failing slopes in problematic cohesive soils.

TABLE 7: COST SAVING AND ENVIRONMENTAL BENEFITS OF EK SLOPE STABILISATION COMPARED WITH OTHER TREATMENT METHODS

EK SLOPE PROJECT	ALTERNATIVE SOLUTION	COST SAVING %	REDUCTION IN EMBODIED CO_2 %
Network Rail, London	Gabion baskets and regrade[1]	26[3]	47[3]
A21, Kent	Soil nailing[1]	29[3]	40[3]
M5, Worcester	Soil nailing[2]	9	43[3]
A419, Swindon	Reinforced soil[2]	35[3]	35[3]

[1]Comparion with remedial solution on adjacent section of failed slope
[2]Based upon value engineering
[3]Meet the objectives of Construction 2015 Industrial Strategy, BIS (2013)

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