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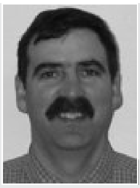
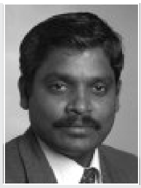
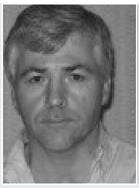


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A case study: delayed failure of a deep cutting in lodgement till

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Recent research on the delayed failure of cuttings in clay clearly recognises and predicts progressive delayed failure of deep cuttings. This is due to a combination of strain-softening, weathering, dissipation of negative excess pore water pressure generated at the time of excavation, and frequent occurrence of prolonged periods of wet weather. There have been several slope failures of this kind in Northern Ireland. This paper discusses a case study based on a failure of a deep cutting, excavated at a slope of 1 in 2, on the A1 near Dromore (County Down) in Northern Ireland. The cutting was in lodgement till, a stiff, heavily overconsolidated clay. The failure occurred approximately 30 years after the cutting was excavated, following a prolonged period of heavy rainfall. An analysis of the failure, together with laboratory test data on soil samples taken from the site, confirmed that by using long-term soil strength parameters the factor of safety of this slope was unity. The conclusion of the analysis is that slopes excavated in this soil should be designed (and assessed) on long-term strength parameters.

NOTATION

B	Skempton's pore water pressure parameter
c'	effective cohesion
c_{qp}	intercept of the failure line in $a : p'$ plane
H	slope of failure line in $q : p'$ plane
k	permeability
p	mean effective stress
q	deviator stress
r_u	pore water pressure ratio
M	slope of critical state line in $q : p'$ plane
σ'_1	effective major principle stress
σ'_3	effective minor principle stress
σ'_h	horizontal effective stress
σ'_v	vertical effective stress
ϕ'	angle of internal friction

1. INTRODUCTION

A large number of roads in the United Kingdom were built during the 1960s and 1970s. Many were constructed through drumlins, which involved creating cuttings in heavily overconsolidated stiff natural clays of lodgement till. Recent evidence[1] suggests that some cuttings are failing after 20–30 years of good performance. Delayed failure can be attributed to a range of factors, including dissipation of negative pore water pressure generated by the removal of the overburden pressure, and the strain-softening nature of heavily overconsolidated lodgement till.[2,3] Consequently the long-term stability of many cuttings in lodgement till is in doubt. A rational mechanism is needed to assess the present state of the slopes to predict future slope stability and, more crucially, to prevent future failures. This poses a considerable challenge to highway authorities, since their remit is to deliver safe and economical transport infrastructure.

There is little doubt that the long-term stability of many slopes in overconsolidated clays decreases with time as water ingress, weathering and dissipation of negative pore water pressures continue to weaken the soil.[2] This effect is exacerbated in situations where drainage is ineffective, and where the factor of safety is approaching unity. In the past few years in Northern Ireland (NI) there has been an increase in awareness by engineers in regard to their responsibility for public safety. This is due primarily to a considerable number of landslides occurring on both the road and rail networks over a relatively short period of time.[4] Little investigation has been carried out into the root causes of such landslides, but it is likely that the above-average yearly rainfall figures over the past few years have been a major contributing factor. Some of the recent landslides include the following.

(*a*) *Glenshesk landslide in 1998*: located on the east coast of County Antrim. The landslide occurred on Monday 2 November 1998, when a 30 m length of Greenans Road at Glenshesk, near Ballycastle, fell into the adjacent glen, leaving a narrow strip of road still crumbling alongside a sheer drop about 15 m deep (Fig. 1). The welfare of 15 families cut off by the landslide was of prime concern, so immediate action was taken to close the road and cordon off the landslide area, and an emergency plan was put into operation. The drainage channel had gradually become blocked over many years, and the water from the upper slope eventually ended up discharging entirely through the sand and gravel deposits beneath the roadway. Five days after the landslide, the filling operation was completed: over 13 000 t of rock armour was removed from the nearby riverbed and placed at road level, allowing the road to be reopened to traffic. Drainage and culvert replacement

Fig. 1. Glenshesk slip

works were completed during the following days, and the road was resurfaced.

(*b*) *Spelga landslide in 1997*. This landslide occurred on a rural road immediately below Spelga Dam, situated in the Mourne Mountains in the south-east corner of NI. The location of the slide caused additional concern because of its proximity to a major dam. Characteristically, the slide occurred following a period of heavy rainfall in the early winter of 1998. The problem was compounded by the fact that the road was cut into a steep hillside in an area of outstanding natural beauty. The road had been cut through a thin deposit of lodgement till and glacial drift, smeared over a granite outcrop sloping at up to 40°. Basic maintenance was not carried out, with the result that it cost more than £1 million to repair the landslide.[5]

(*c*) *Dromore landslide in January 1999*. This slope failure occurred in a cutting near Dromore, some 27 km south of Belfast. The cutting was located on a strategically important route carrying significant volumes of traffic on the main dual carriageway linking Belfast to Dublin. The slope was in a 19 m high cutting at a slope angle of 1V:2H in a heavily overconsolidated lodgement till. This slope failure generated considerable transport disruption, and received adverse media publicity. It is important that lessons are learned from the experiences of dealing with this landslide and, accordingly, this paper discusses the various geotechnical and design considerations considered in the repair of the landslide.

2. SEQUENCE OF SLOPE FAILURES AT DROMORE

The cutting is located near Dromore on the A1 (Fig. 2). It was excavated to a slope of 1 to 2 (27°) in 1972, and is approximately 19 m high. It is not clear whether or not a detailed slope stability analysis had been carried out at the time of the excavation. It is likely that the original excavated slope was based on the simple 1V:2H rule of thumb widely used for cuttings in stiff clays at that time. The cutting first failed in January 1999 as a result of a large rotational slide developing (Fig. 3). A plan showing the extent of the failure is shown in Fig. 4. The failure occurred after several months of significantly higher-than-average monthly rainfall levels, and at the end of several days of exceptionally heavy rainfall: 41 mm of rain fell on 30 December 1998.

A large tension crack was visible close to the crest of the cutting (Fig. 5), and the toe of the landslide revealed itself as a distinct bulge, emerging in the inside lane of the dual carriageway (Fig. 6). The hard shoulder was raised approximately 0·5 m and, as a result, the inside lane of the dual carriageway adjacent to the landslide had to be closed immediately. The estimated position of the slip surface can be seen in Fig. 7(a). This estimated slip surface is based largely on the geometry of the tension crack and the distortion of the carriageway, but was confirmed by stability analysis.

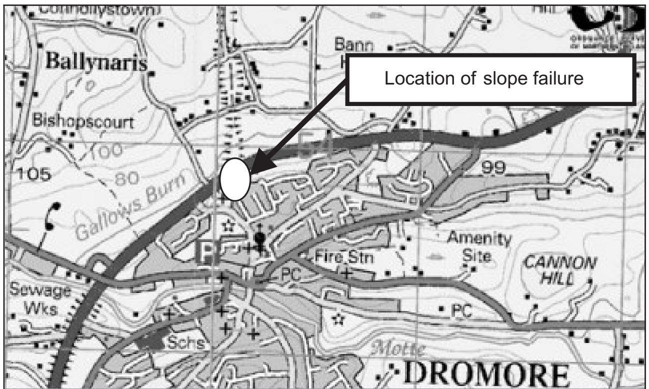

Fig. 2 Location and plan of slip failure area near Dromore, Co. Down; OS licence no. 700714 (scale 1:20 000)

Fig. 3. Original slope failure of cutting in January 1999: note carriageway heave

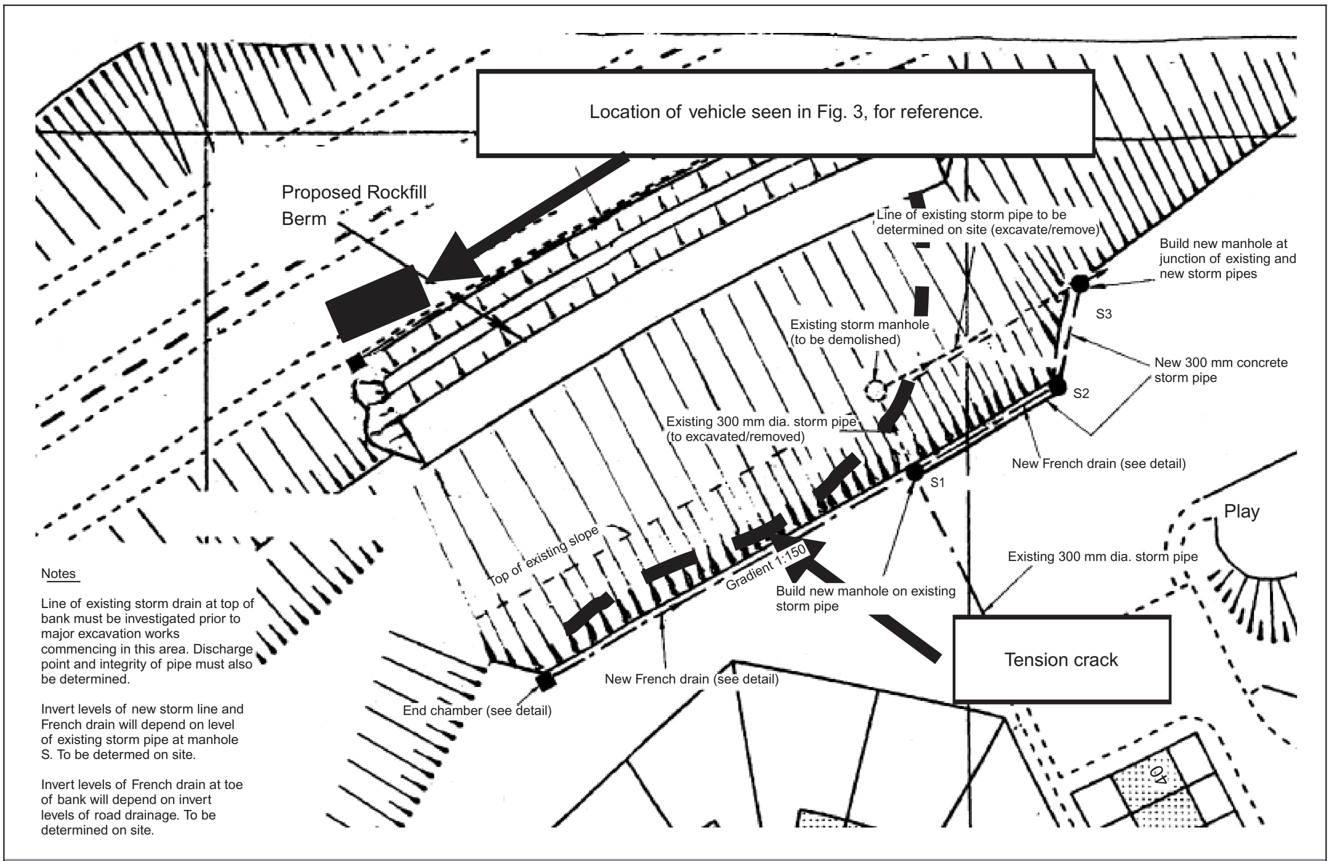

Fig. 4. Plan drawing of temporary repair and location of tension crack (scale 1:20 000)

Fig. 5. Tension crack at crest of cutting

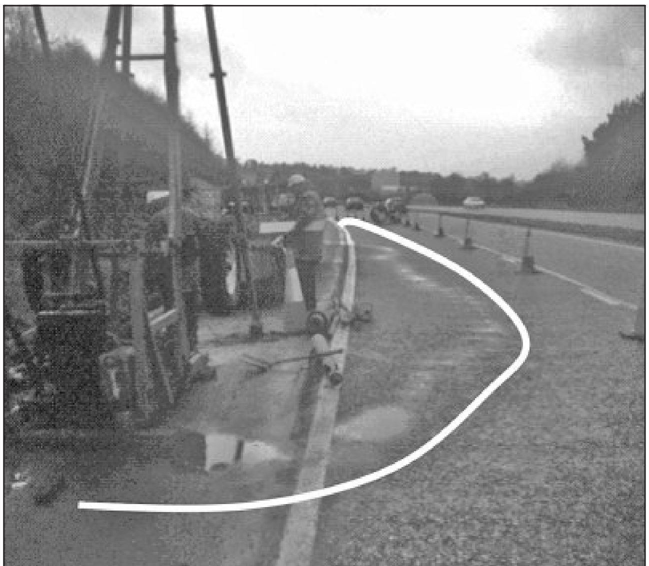

Fig. 6. View south on southbound carriageway: failure surface emerging at the toe, causing heave in the hard shoulder and inside lane of dual carriageway

A temporary remedial design was proposed by Construction Service: it consisted in regrading the cutting to a slope of 1V to 2·4H (23°) and providing a counterweight berm strategically positioned at the toe. This was carried out as an interim measure to prevent further disruption to traffic flows and minimise further distress to the carriageway. These temporary remedial works were completed in August 1999, and the cross-section of the regraded slope is shown in Fig. 7(b). However, owing to various constraints, such as horizontal sightline requirements for the carriageway and land ownership problems, the work undertaken did not fully reflect the original design recommendations. The berm was undersized because it had to be ‘trimmed’ down significantly to satisfy safety regulations, that is, driver visibility criteria. Furthermore, the

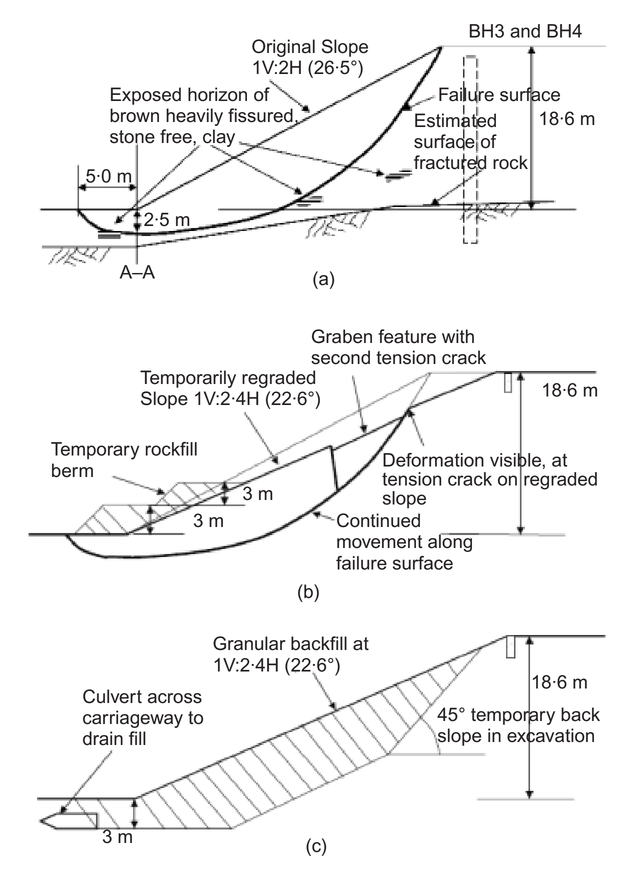

Fig. 7. Sequence of events of failure, temporary and permanent repair: (a) estimated shape of failure surface (January 1999); (b) regraded slope with temporary berm and Graben after further movement (October 2000 to March 2001); (c) final repair—excavation and granular fill (December 2001)

slope was not regraded fully to the specified design gradient because permission could not be secured from the local authority to remove part of a recreational area/playing pitch at the crest of the cutting. Emergency drainage systems were also implemented at the crest and toe of the slope by excavating trenches up to 1·5 m deep and filling with coarse aggregates.

Subsequent movement of the temporarily repaired slope was recorded in October 2000, following a further period of heavy rainfall. Finally, on 6 November 2000, following another period of exceptionally heavy and prolonged rainfall—approximately 40 mm in 24 h—the slope moved significantly, and a Graben feature formed (see schematic in Fig. 7(b) and photograph in Fig. 8). One lane of the dual carriageway was again closed to traffic. A further large temporary berm, extending well into the existing carriageway, was placed at the toe of the slope to increase the stability and prevent catastrophic failure. This is visible in Fig. 8.

3. GEOTECHNICAL INVESTIGATION

Following the initial landslide in January 1999, Construction Service carried out a further and more detailed site investigation. Four borings were carried out, and the samples obtained were tested in three laboratories: Construction Service, Soil Mechanics Ltd, and the School of Civil Engineering at the Queen's University, Belfast. Undisturbed samples were obtained in U4 tubes and a series of disturbed

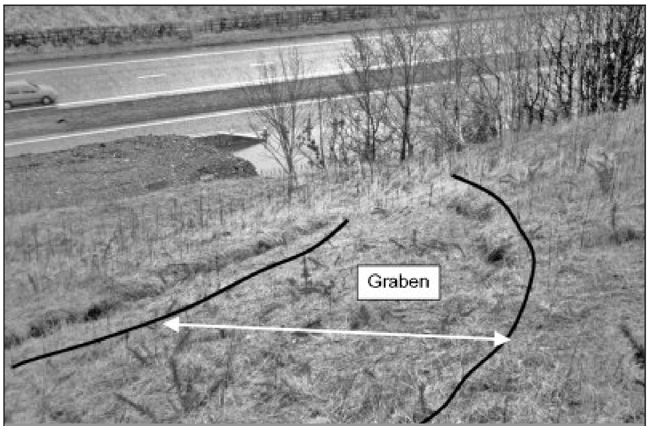

Fig. 8. Graben feature and temporary berm at toe of slope

samples were collected in bags. A detailed laboratory characterisation was carried out on samples obtained from boreholes 3 and 4 located near the crest of the slope (Fig. 7(a)). A list of characterisation tests performed on the samples from these boreholes is presented in Table 1. The majority of the samples consisted of lodgement till, a glacial deposit laid down and consolidated under an ice sheet, probably during the late Devensian Stage of the last Ice Age, to form what we now call a drumlin. Some of the samples, as listed in Table 1, showed evidence of periglacial weathering. Some fill materials and samples of stiff laminated/fissured clay were also encountered, although not highlighted in the original interpretation of the boreholes. This layer may have influenced the development of the failure, and is discussed later.

3.1. Characterisation

Standard laboratory tests were performed on selected specimens to characterise the lodgement till. These included density measurements, particle size analysis and index properties. The predominant clay mineral is probably illite, which appears to be typical of NI clays. The density of the clay fraction varied from 2275 kg/m^3 to 2295 kg/m^3, and moisture contents varied from 12% to 15%. The amount of clay varied slightly from the top to the bottom of the deposit. Approximate percentage contents by weight of clay, silt, sand and gravel (stones) were 12%, 22%, 23% and 43% respectively. The individual grading curves were also approximately linear, and thus conform to the suggestion[6] that a 'straight-line grading' is characteristic of lodgement tills.

In general, it appears to be standard practice in some commercial site investigation firms to determine moisture contents from measurements taken on large triaxial specimens, typically 100 mm in diameter and 200 mm long. The moisture contents reported in Table 1 were determined in this way. However, in certain circumstances the properties of particular interest must be related to the matrix of the deposit or soil itself rather than the entire sample as a whole. This is particularly important if large particles are isolated within any given stratum, and therefore do not contribute significantly to the collective strength of the material. Liquid and plastic limits were determined on samples of soil from which all particles larger than 425 μm had been removed. The results show that all points lie close to but above a line parallel to the Casagrande A line. This line has also been termed the T-line[7,8] and, significantly, most NI subsoils satisfy this condition.

Depth: m	Moisture content	Atterberg limits			Particle size: %				Moisture content*	Lab
		LL	PL	PI	Clay	Silt	Sand	Gravel		
1·00–1·20	32·2	58	49	9						DOE
1·20–1·40	37·4				0	16	23	61		DOE
1·80–2·00	15·3	36	19	17						
2·00–2·20	13·4				14	19	29	38	21·6	DOE
2·25–2·35		39	20	19						
2·70–2·90	11·8	44	21	23	13	15	30	42	20·3	DOE
3·50–3·95	8·7	39	17	22	14	22	21	43	15·3	SML
3·95–4·05	13·7	37	18	19						DOE
4·50–4·95	11·0	30	14	16	15	22	25	38	17·7	SML
4·95–5·05	1·59	36	17	19						
5·50–5·95	9·5	41	16	25	13	17	16	54	20·6	SML
6·50–6·59	12·6	34	16	18						QUB
6·95–7·05	12·1	39	18	21						
8·50–8·59	10·9	36	18	18						QUB
9·50–9·95	12·0	37	18	19						QUB
10·50–10·95	12·0	34	16	18	15	24	25	36	18·7	SML
11·50–11·95	12·0	36	16	20	15	27	31	27	16·4	SML
11·95–12·05	12·7	39	18	21						
12·50–12·90	15·0	39	18	21	14	28	28	30	21·4	SML
14·00–14·45	12·0	39	16	23	13	24	26	37	19·0	SML
15·00–15·45	15·0	38	17	21	15	38	26	21	19·0	SML

LL, liquid limit; PL, plastic limit; PI, plasticity index; DOE, Department of Environment soils laboratory; SML, Soil Mechanics Limited soils laboratory; QUB, Queen's University of Belfast soils laboratory.
*Moisture content following removal of coarse material

Table 1. Borehole 3: soil classification tests

Again, the results are remarkably consistent, with average liquid and plastic limits (LL and PL) of approximately 38% and 18% respectively and a plasticity index (PI) of 20%. On this basis, the lodgement till can be classified as an inorganic clay of medium plasticity.

Inspection of Table 1 shows that the average moisture content of the lodgement till is significantly less than the moisture content at the plastic limit. This is due to the fact that the moisture contents have been determined on samples containing stones, whereas stones and other coarse particles have been removed in the determination of the plastic limits. In order to compensate or at least make an approximate correction for this effect, moisture contents have been adjusted where sufficient data are available. This has been carried out by estimating the weight of gravel-sized stones in each sample in order to back-calculate the actual mass of the clay fraction. The corrected moisture contents (marked *) are given in Table 1. Note that the effect of this correction is not trivial, because it leads to an increase in moisture content of about 50% across the board. The end result, generally speaking, is that moisture contents of the clay matrix are slightly greater than the moisture content at the plastic limit.

3.2. Undrained compression test

A large number of consolidated undrained tests were performed on samples 105 mm in diameter by 200 mm long. Six samples obtained from borehole 3 were subjected to single-stage loading under undrained conditions, and a further two samples from the same borehole, taken at depths of 10·5 m and 11·55 m, were subjected to stage loading (multiple shearing stages) under undrained conditions. Two further two samples obtained from borehole 4 were also subjected to stage loading.

The samples were initially saturated using the standard procedures.[9] Nearly complete saturation of the sample was ensured by achieving a B value of more than 0·95. The samples were sheared at strain rates varying from 0·08% to 0·12% per hour. The stress–strain response of the samples when subjected to undrained compression is typical of heavily overconsolidated clays. The samples were taken though fairly large axial strain, though none of the samples reached true critical state.

Figure 9(a) shows the deviator stress plotted against mean effective stress for single-stage tests performed on six samples obtained from borehole 3. These tests were performed by Soil Mechanics Ltd, and the test data were analysed to estimate the strength parameters at failure (peak) and critical state. The failure state was identified by locating the points on the stress paths in the $q:p'$ plane where the stress ratio, σ_1'/σ_3', reaches a maximum value, and they are shown in Fig. 10(a) (the solid points represent the data obtained from single-stage tests). As stated previously, none of the samples reached critical state, even at large axial strains. Accordingly, the stress states of the samples at the end of the tests were considered a reasonable representation of the conditions prevailing at the critical state, as shown in Fig. 10(b).

Although the samples were obtained from within the same deposit, the previous stress history of each sample varies, depending on the depth at which it was taken and its state at the time of removal. It is assumed in the present slope stability analysis that the cohesion is constant and does not vary with

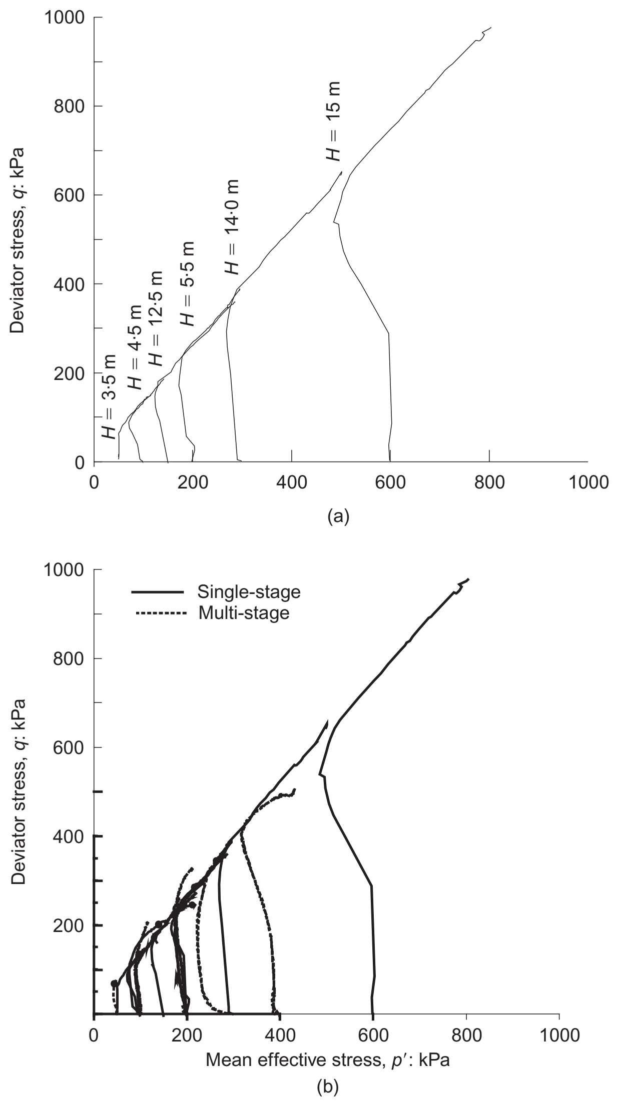

Fig. 9. Stress paths of (a) single-stage and (b) multi-stage loading on samples

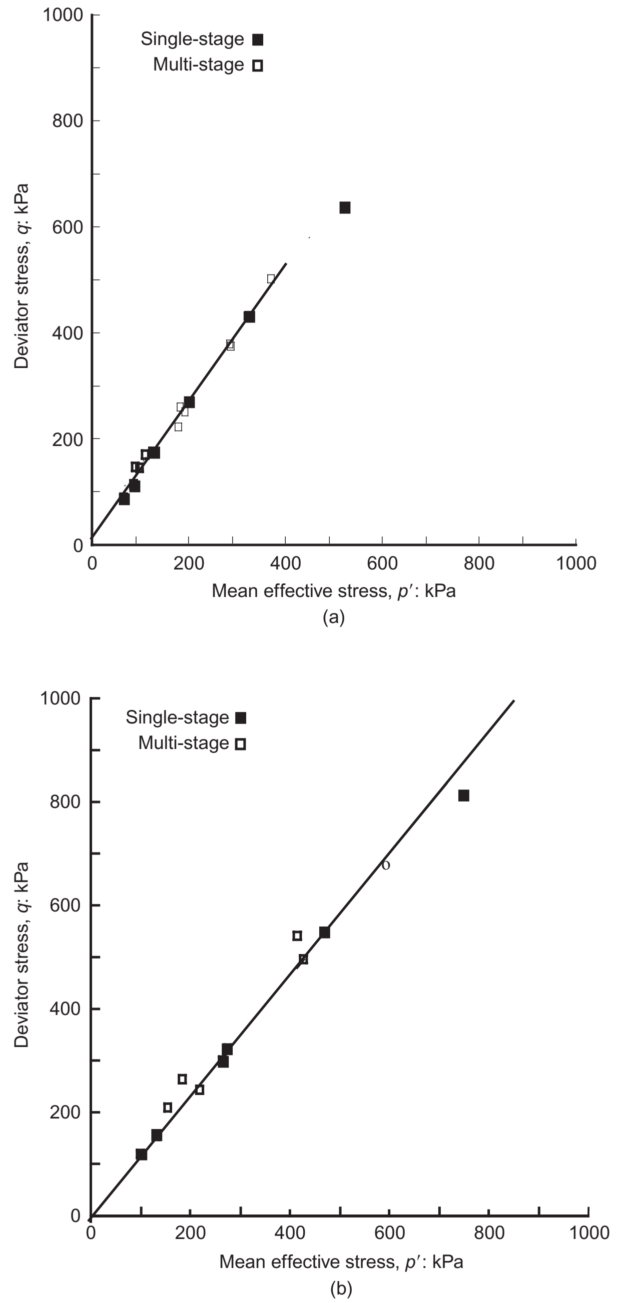

Fig. 10. Conditions at: (a) failure (peak stress ratio); (b) critical state

depth. This is a reasonable assumption, as the present analysis is generally focused on the critical state of the soil. Testing samples from various depths can only give an average value of c'. To enable an accurate measurement of this soil parameter, testing should be performed on samples obtained at the same depth. This can be achieved by testing smaller samples (38 mm diameter), though this may not necessarily be possible in lodgement till since it contains large stones. This situation can possibly be avoided by testing 105 mm diameter samples through staged loading. In the stage tests, samples were subjected to at least three stages or phases of loading. The samples were initially consolidated to three consolidation pressures, equivalent to $\frac{1}{2}\sigma'_v$, σ'_v and $2\sigma'_v$, where σ'_v is the original effective vertical pressure. Consolidation was followed by undrained compression, and in the first two stages of compression the loading was terminated when the stress ratio reached a maximum value. However, the disadvantage of using this method is that it is not easy to predict at what axial strain the peak stress ratio will occur. If the sample has experienced plastic shear strains during the first two stages of compression, it will inevitably alter the structure of the soil and, therefore, the stress–strain behaviour of the soil. This could have a detrimental effect on the strength properties of the soil. Fig. 9(b) shows the stress paths obtained for stage tests performed on samples obtained from borehole 3; also included are the stress paths obtained from single-stage tests. It appears that the position of the failure line is not particularly affected by the stage loading, as it yielded results similar to those of single-stage loading. For the estimation of the strength parameters, data obtained from single-stage and multi-stage loading were considered. The slope of the failure line, H (Fig. 10(a)), is approximately 1·29, and the intercept c_{qp} is approximately 13 kPa. These values correspond to an angle of internal friction of 32° degrees and cohesion c' of approximately 4 kPa. However, the effective cohesion of tills, particularly those found in Ireland, are very low, and in some cases insignificant.[10] The slope of the critical state line Fig. 10(b), M, is approximately 1·18, and this corresponds to an angle of internal friction of 30°. There appears to be no significant difference between the angle of internal friction at peak and that at critical state, though for the purpose of analysis an angle of internal friction ϕ' of 31° and a cohesion c' of 1 kPa were adopted. This small amount of cohesion was included to allow for progressive failure, and such failures are particularly prevalent in glacial till, where the clay is the predominant material. Also note that the cohesion c' of 1 kPa is somewhat lower than the measured value. However, soil along the failure surface was not exposed to uniform straining, and therefore an assumption was made to analyse the slope at conditions close to critical state.

4. SLOPE STABILITY ANALYSIS OF CUTTING

Back-analyses have been carried out on both the original cutting and the temporarily repaired cutting. The analyses were carried out using the GeoSlope software Seep/W and Slope/W. Seep/W allowed steady-state pore water pressures to be calculated in the slope for given boundary conditions. Slope/W uses limit equilibrium methods to analyse the stability of a slope, and interacts by using effective stresses in the slope generated from pore water pressures determined by Seep/W. The modified Bishop method was used throughout these computations in order to maintain consistency.

The back-analysis technique requires an assessment of the groundwater regime, though little information on the water table was available at the time. Consequently, a conservative but reasonable estimation of the likely groundwater flow pattern and phreatic surface was made. Note that the back-analysis is carried out to give an indication of the in situ soil strength properties. Furthermore, it can be used to compare these values with laboratory test results in order to increase the level of confidence in the estimated shear strength parameters of the soil. This allows the engineer to have greater confidence in applying various remedial measures and techniques, such as the installation of drainage and/or the placement of toe loading.

In estimating the groundwater regime it should also be noted that, by assuming an onerous seepage condition at the time of failure, a high soil strength value will be predicted, giving a factor of safety of 1 in the back-analysis. Conversely, by assuming a less onerous seepage regime, a proportionally lower value of strength will be predicted in the back-analysis. It is the combination of the three variables—pore water pressure, soil shear strength, and knowing the precise form, geometry and position of the slip surface—that makes the back-analysis more accurate. It is for this reason that the investigation of the stability of slopes is made more certain with good-quality groundwater information, accurate soil properties, and a detailed post-failure site investigation.

4.1. Back-analysis of original failure

Figures 4 and 7(a) show, respectively, plan and sectional views of the original slope. The site investigation revealed that the upper horizon of the Silurian shale bedrock underlying the slope was highly fractured, presumably broken up by ice action and, therefore, significantly more permeable than the underlying intact rock. Standpipes were then inserted to the weathered rockhead at the toe of the cutting and head levels were monitored. These measurements, however, failed to produce any evidence that might suggest the existence of a continuous free-draining layer at rockhead level. The existence of such a permeable layer could have a significant effect on the stability of the slope, and, for this reason, the problem was examined in some detail. Various possible hypotheses were considered, but it was finally concluded that the existence of a 'free draining' layer at the base of the lodgement till had no significant effect on the stability of this slope.

There is a considerable degree of uncertainty about several aspects of the drainage of the slope. Prior to the landslide occurring, the site was considerably overgrown, and it is now not possible to assess the effectiveness or otherwise of any positive drainage measures that were in place at that time. No information is available in relation to the drainage installed during the construction of the play area at the top of the slope. The value of permeability k assumed is extremely low (5×10^{-11} m/s), and therefore no attempt has been made to consider the effect of a reduction in permeability with depth. Ponding was observed at the top of the slope during the site investigation, and this is the only direct evidence that supports the assumption of full saturation.

Figure 11 illustrates the steady-state seepage conditions, based on the assumptions of infiltration at ground surface at the top of the slope, runoff on the actual slope, and an impermeable bedrock basal boundary. The groundwater flow is approximately parallel to the slope, with corresponding pore pressure ratios (r_u) in the region of 0·4 to 0·5. Clearly, these are stringent drainage conditions, which to a large extent reflect the impervious rock surface at shallow depth at the toe of the slope. Based on the shear strength parameters of $c' = 1$ kPa and $\phi' = 31°$, the calculated minimum factor of safety is 0·99, corresponding to the circular slip surface or failure plane shown in Fig. 12. This

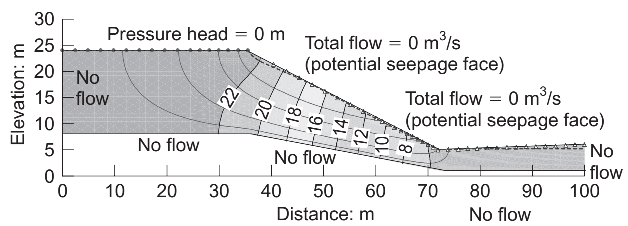

Fig. 11. Seepage condition: original slope

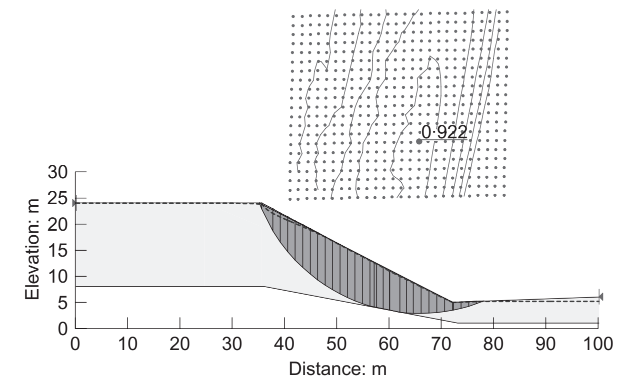

Fig. 12. Slope stability analysis, original slope: $c' = 1$ kPa, $\phi' = 31°$

critical slip surface emerges near the top and bottom of the slope, and is therefore in general agreement with the actual failure surface. The fact that the minimum factor of safety is very close to unity is, to some extent, fortuitous, because the inclusion of a tension crack and/or the use of a different method of analysis would have resulted in an even lower factor of safety. Nevertheless, these computations confirm that the strength parameters obtained from laboratory testing, combined with the assumed groundwater regime, provide a clear and consistent picture of the mechanism of the original failure.

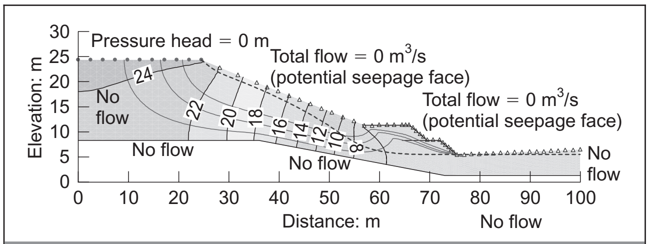

Fig. 13. Seepage condition: repaired slope

4.2. Analysis of stability of temporarily repaired slope

The temporary remedial measures proposed by Construction Service consisted of a general reduction in the slope gradient to 1 in 2·4 (23°), together with the construction of a berm of granular material at the toe, as shown in Fig. 7(b). In addition, drainage to a depth of 1·5 m was included at the crest, along with the reinstatement of drainage along the hard shoulder of the carriageway at the toe of the slope. A walkover survey in October 2000 revealed no visible evidence of the crest drain, as the top of the slope had become heavily overgrown with vegetation. It seems unlikely that this drain prevented surface runoff from the hardstanding play area above from running onto the face of the slope and entering the tension crack.

The groundwater regime incorporating the 'as constructed' remedial measures and making use of the same assumptions, as discussed above, is shown in Fig. 13. Comparing this with Fig. 11, it can be seen that the remedial measures had little effect insofar as drainage is concerned, with the values of pore water ratio remaining in the range 0·4 to 0·5.

As the movement of the temporarily repaired slide was essentially a reactivation on the original slip surface, the residual shear strength of the till was examined. During the excavation of the repair, when all the till to a depth below the basal slip plane was removed, a layer of heavily fissured brown, stone-free clay was observed. This layer was between 100 mm and 500 mm thick, and the slickensided basal slip plane could clearly be seen having formed for some length in this layer. The approximate location of this heavily fissured clay is shown in Fig. 7(a). It appeared as if this layer departed from the basal slip plane at some depth approximately vertically below the crest tension crack and continued into the drumlin. There is some conjecture in the literature that these fissured layers are due to tectonics or glacial-induced shearing in the drumlin during the re-advance of ice sheets or glaciers.[11,12] Fig. 14 shows a large triaxial sample taken across the basal slip plane in the heavily fissured clay. The slickensided surface of the slip plane can be clearly seen in the left sample, along with striations showing the direction of relative movement. The right sample is the inverted match of the left sample, and the slickensided slip plane can be seen at the top of this sample.

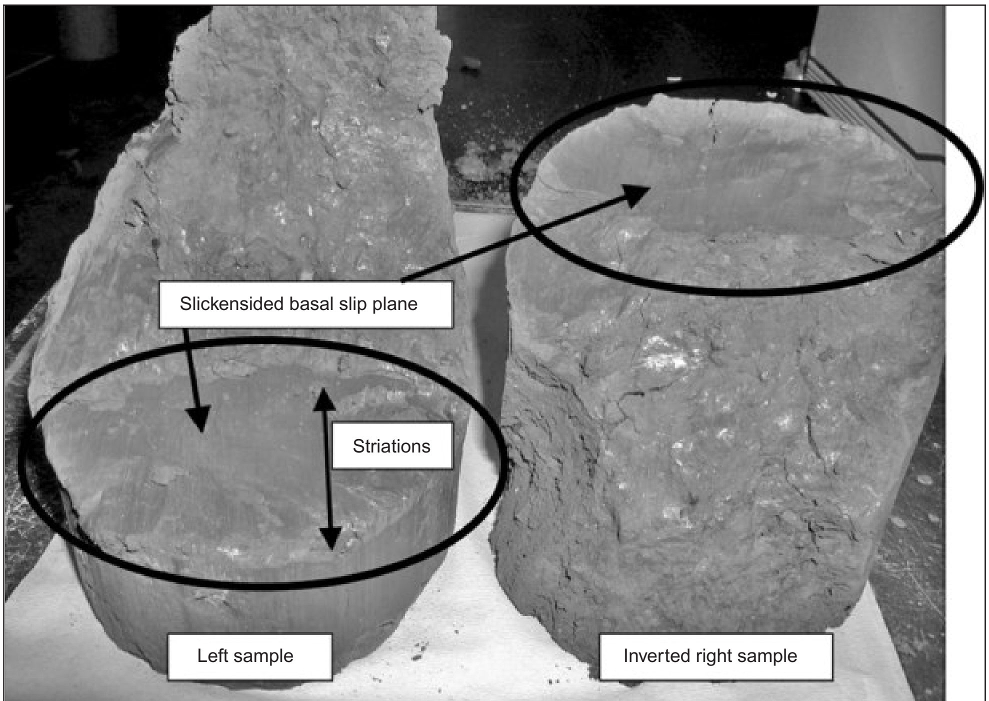

Fig. 14 Heavily fissured stone-free brown clay, showing slickensided basal failure plane obvious in left sample. Right-hand sample taken from same block as matching left sample but inverted

Extensive residual shear strength tests using a ring shear apparatus were subsequently carried out on this heavily fissured clay,[13] and gave a residual shear strength of $\phi' = 18°$, with plastic and liquid limits of 32% and 60% respectively. A stability analysis based on this residual shear strength gave an estimated factor of safety of this slope of approximately 1·0, as shown in Fig. 15.

Following costing of various remediation solutions, including soil nailing, a reticulated pile wall and deep drainage, the decision was made, based on cost and risk, to repair the slope by excavating the material to below the failure surface and replace it with free-draining crushed rock.

4.3. Assessment of geotechnical risk in cuttings

The need for risk assessment of geotechnical assets, including cuttings, on the road network arises principally as a means of satisfying the Roads Service's responsibilities

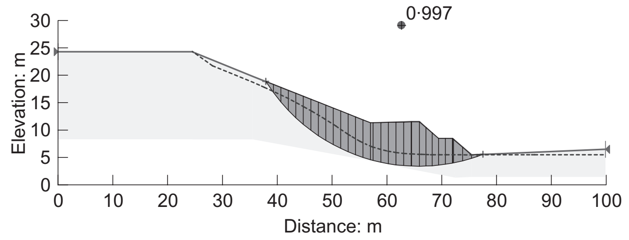

Fig. 15. Slope stability analysis: temporarily repaired slope, residual strength parameters $c' = 1$ kPa, $\phi' = 18°$

under relevant statutory provisions. They included the general duties laid down in the Health and Safety at Work (NI) Order 1978[13] and the more specific duties in the Management of Health and Safety at Work Regulations (NI) 2000.[14] The Management of Health and Safety at Work Regulations (NI) 2000 impose a statutory duty on employers to conduct regular risk assessments. This is to ensure that safety is not degraded, and that risk is maintained at a value that is as low as is reasonably practicable. Risk assessment is therefore a tool that can be applied by maintenance engineers to ensure that the expenditure of funds is justified and allocated in response to safety and business needs. Most authorities managing geotechnical infrastructure have asset management procedures that encompass risk assessment procedures. It is clear from the detailed geotechnical analysis of the failure at Dromore and the subsequent laboratory testing that the stability of cuttings in lodgement tills excavated at a slope of 1 in 2 (27°) has factors of safety close to unity, particularly under adverse drainage conditions such as those encountered at Dromore.

It is clear that soils in cuttings, when subjected to long periods of wetter-than-average weather, will reduce in strength (owing to the process of weathering or softening) and, as a consequence, the factor of safety of such cuttings will reduce with time. The existing evidence for long-term reduction in shear strength of clays due to dissipation of negative pore water pressures is, at present, based largely on research on stiff clays with a higher clay content than that observed in 'local' tills, that is, generally clays in the south-east of England. There are few existing data on the long-term change (or reduction) in in situ soil strength with time for stiff lodgement tills in cuttings, although it is very likely that these soils behave in a similar manner and have many similar characteristics. Based on the analysis of the failure at Dromore, it is likely that cuttings of a similar nature in NI have low factors of safety in regard to stability. The soil strength of these cuttings is, at best, remaining constant but, more likely than not, is probably reducing with time.

Combining the factors of over-steepened cuttings, the likely reduction in the long-term strength of clays in such cuttings, and the forecast increase in average rainfall,[15] it is strongly recommended that further research be carried out in this area. More detailed assessment of other vulnerable slopes is deemed critical in this respect, and there should be a systematic approach to the risk of failure and the maintenance and assessment of cuttings on the road network.

5. CONCLUDING REMARKS

It is apparent that the landslide in the cutting at Dromore was caused by long-term reduction in strength of the soil due to progressive failure, strain-softening, and the dissipation of negative excess pore water pressures generated during excavation of the cutting. The landslide was triggered by the exceptionally heavy rainfall in the preceding days and the significantly wetter-than-average winter.

A layer of heavily fissured stone-free brown clay found to be coincident with the basal slip plane may also have contributed to the instability of this slope. Although some evidence of this layer was found in a preliminary site investigation, the layer was largely ignored in the original analysis. It is likely that this clay-rich layer reduced the overall mobilised shear strength along the failure plane and contributed to the failure.

Based on general principles it may be stated that slopes of the order of 1 in 2 in glacial till under adverse drainage conditions have an inadequate factor of safety unless positive drainage conditions are included in the design. Design slopes recommended in *Engineering in Glacial Tills*[16] are considerably flatter than the 1 in 2 slopes common in cuttings in NI. It is likely that other cuttings of a similar age and slope angle in soils similar to the cutting at Dromore will exhibit progressive long-term reduction in stability. The overall stability of cuttings in lodgement till is adversely affected by poor positive drainage—both subsurface soil drainage and surface water runoff. Greater emphasis should be placed on the provision of deep positive drainage in slopes and at the crest and toe of slopes. Considering the cost implications for the repair at Dromore, it is clearly significantly more costly to repair a failed slope than to improve the factor of safety of an intact slope.

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