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Title of paper: Analysis of the peat slide at Pollatomish, County Mayo, Ireland

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Abstract: A major landslide event occurred at Pollatomish, County Mayo, Ireland in September 2003, during a period of intense rainfall. It comprised about 30 significant individual longitudinal planar type slides of peat and weathered rock. Relatively simple limit state stability analyses, using the method of slices and an infinite slope analysis, were used to model the slide and it was found that the features observed on site could easily be reproduced. These included confirmation that thin layers of peat could be stable on steep slopes but the margin of safety reduces rapidly under elevated pore pressure conditions. As was observed in the field, the analyses suggested the most vulnerable zone was the upper layer of weathered rock but that slides could occur in the peat if its thickness was appreciable. Careful site characterization is vital in such studies. Here efforts have been made to understand the effect of fibres on the peat strength and some sensitivity analyses have been performed to assess the critical engineering parameters of the peat.

Key words: peat; weathered rock; translational (planar); County Mayo; Northwest Ireland;

Introduction

A major landslide event occurred in the Pollatomish area of North County Mayo, Ireland on the night of Friday 19th September 2003, during a period of very heavy rainfall. The many landslides resulted in considerable damage to roads, bridges and property and led to the evacuation of over 40 families. To date approximately €3.8 million has been spent on repair of the damage caused by the event and in compensation to those who suffered loss. It comprised about 50 individual longitudinal planar type slides of peat and weathered rock, with individual slides involving between 15 m³ and 20,000 m³ of material.

Peat slides, bog slides, flows and bursts are not unusual in Ireland with the earliest documented peat slide having occurred in 1697 (Feehan and O'Donovan, 1996). These authors provide an excellent overview of the subject and detail some 38 events in Ireland. Warburton et al. (2004) also review the topic and conclude that the understanding of local drainage and hydrological conditions are essential in order to understand peat slides but that “the prediction of the location and timing of such events is a long way off” (page 139).

Detailed descriptions of the event and the prevailing conditions have been given by Creighton and Verbruggen (2003), Tobin (2003) and Duggan (2004) and only a short summary will be presented here. The main objectives of this paper are to report on some:

- geotechnical testing of the peat and weathered rock from near the site,
- engineering analyses of the slide.

It is hoped that this work may contribute to the overall understanding of this topic and assist in hazard risk assessment of potential future slides in similar conditions.

Description of location of slides

Location and topography

The landslides that are the subject of this paper occurred in the Dooncarton Mountain area of Pollatomish in North Mayo, on the west coast of Ireland, some 12 km east of the town of Belmullet and 55 km north of the town of Westport (Figure 1). Dooncarton mountain ridge dominates the topography of the area. Its summit height is about 260 m above sea level and the land drops steeply on all sides, towards the coast on the east and north sides and to the interior river valley on the south side of Dooncarton Mountain.

The slope profile (applies generally but particularly to the north and east facing hills) comprises some or all of the following elements, as shown on Figure 2:

- Ridge line.
- Upper slope. This is the zone generally above the 150 m contour and comprises a steep upper slope with inclination 30° to 60° . It may represent an old corrie back-wall where there has been accentuated erosion by ice during the last Ice Age.
- Middle slope. Relatively smooth slope of 10° to 30° between the 50 m and 150 m contours.
- Lower slope. Towards the coastal lowlands the slope increases to approximately 45° to 60° . It has a benched rather than smooth appearance
- Coastal strip. This consists of a narrow strip of moderately sloping ground varying from 10° to 30° .

Geology and ground conditions

According to Long et al. (1992), the local geology consists of metamorphic rocks some 400 million years old. The upper slopes comprise relatively easily weathered schists.

Schists and quartzites make up the middle slopes and the lower slopes are underlain by hard metamorphosed sandstones. The different hardness of adjoining formations has resulted in the stepped profile of the mountain. Some faulting is present in the area including one which forms part of the stream flowing into the bay at Pollatomish. However these faults have been tectonically stable for at least 200 million years.

An important characteristic of these rocks is that they are all highly impermeable, being tightly cemented and compact.

In the area of Dooncarton Mountain and the village of Pollatomish there is only a thin cover of glacial sediments and indeed many areas are drift free. Synge (1968) proposes that the upper parts of Dooncarton Mountain were never glaciated, the icesheet only reaching a certain height, defined as the drift limit. Creighton and Verbruggen (2003) suggest that the drift limit in this area is somewhere between 50 m and 100 m above sea level.

A layer of weathered rock lies above the intact bedrock. Its thickness ranges from 0.2 m to several metres. Locally, weathered rock is absent, particularly in the upper and lower slopes, see Figure 2. The sandy nature of the weathered rock reflects the lithology of the underlying quartzites and schists. There is an absence of residual structure in the material, particularly in the upper slopes, suggesting it has been affected by mass movement. There is some evidence for a preferred down slope orientation for the gravel fragments in the weathered rock. It is likely this is as a result of a freeze – thaw mechanism (solifluction) causing down slope creep.

Blanket peat covers most of the slopes. Its thickness varies between 0.2 and 2 m with an average of about 1 m. Humification of the peat at its base varies between H4 to H5

(somewhat to moderately humified) according to the Von Post scale (Hobbs, 1986) Fibres are readily identifiable, are a mixture of coarse and fine, and occur at random orientation. Close to the peat / weathered rock interface, and generally located 200 mm to 300 mm below the top of the weathered rock, there is a layer of “hard pan”. As will be described below, this layer was observed just below some of the failures. This is considered to be a precipitated layer, of iron mainly, but also of manganese minerals, leached from the overlying deposits. Its thickness ranges from a few millimetres to 100 mm and comprises nodular black amorphous silt / clay which is cemented to angular gravel or cobble sized fragments, see Figure 6. Its state was not always as well developed as shown on Figure 6 and sometimes only an orange colour staining was visible in the weathered rock. An important characteristic of this layer is its low permeability and its resistance to root growth. Above the hard pan is a layer of irregular thickness typically 50 to +100mm, but can be absent, of soft to very soft brown to dark brown slightly sandy silt/clay. The top of the zone comprises a thin layer (typically 3 to 5 mm) of stiff black to dark brown amorphous silt/clay (referred to as dopplerite by Feehan and O’Donovan, 1996). The thin layer is polished on the upper surface, which may represent slickensides, see Figure 6.

Drainage

A number of natural minor stream drain the upper slopes. These are typically narrow gullies less than 1 m wide with the stream bed located in weathered rock. Further downstream these streams are intercepted by a ditch which runs transversely to the slope and forms the boundary between commonage upslope and privately owned land downslope, see Figure 2. Several larger stream cut through the ditch and flow down slope

into the sea. Much of the rainfall and resulting landslide material was channelled into these streams.

The impermeable nature of the “hard pan” and bedrock impedes downward migration of groundwater. The zone immediately above the “hard pan”, within the weathered rock or the basal peat represents a preferential drainage path to groundwater movement downslope. Above the “hard pan” within the weathered rock and peat natural conduits are present. These pipes range in diameter from 0.1 m to 1m and are generally aligned down slope. Murphy (2004) describes the successful use of ground penetrating radar (GPR) techniques to identify these pipes at the location of a slide on an east facing slope on the south side of Dooncarton mountain.

Description of the slide event

Previous slope instability on Dooncarton Mountain

A peat slide occurred on the southern slopes of Dooncarton, below the 250 mOD summit, in the late 1970s. Inspection of this slide showed failure of peat on mineral soil. Further anecdotal information suggests a peat slide occurred on the northern slopes of Dooncarton during the last 20 years. There is a history of peat cutting for fuel in the area and this may have contributed to the slides.

Weather conditions

Prior to the slides the weather during the summer of 2003 was exceptionally dry and warm. The Irish Meteorological Service, Met Éireann, report that the maximum temperature ever recorded at Belmullet (27.7° C) occurred in early August 2003. Rainfall for May, June and July 2003 was well below the 30 year average of about 195 mm. However rainfall for August 2003 was close to the monthly 30 year average of 94 mm.

Total rainfall for Belmullet for 2003 was 1075 mm which was about 94% of the 30 year average. Nonetheless soil moisture deficit at Belmullet rose steadily through August 2003 and had reached 54 mm by September 4th.

Intense and localised rainfall fell on the area between about 8:45 pm and 10:45 pm on 19th September. The exact amount of rainfall is not completely clear but it is unlikely to have been less than 80 mm (Tobin, 2003).

Failures elsewhere in same period

During the early morning of the same day, landslides also occurred on the southern side of the Shetland Islands. A series of peat slides damaged 8 km of the main A970 road, south of Lerwick. The slides caused the collapse of a bridge, the breaching of several water and sewage mains as well as the destruction of several cottages and farm buildings (Benfield Hazard Research Centre, 2003). Some sheep were killed but there was no loss of human life. Like in the majority of cases in Pollatomish failure was in the mineral soil underlying the peat.

According to Tobin (2003), radar images confirmed that the same storm affected both West Mayo and the Shetland Islands. A similar amount of rainfall, i.e. about 79 mm, is thought to have fallen in the Shetlands in a short period over a localised area (data from Met Éireann reported by Tobin, 2003). The similarities between the intensity and localised nature of the two events in Shetland and Pollatomish, which occurred hours apart, is striking and is indicative of particular atmospheric instability on that day.

It is also interesting to note that about 3 weeks earlier, on 26 August 2003, a peat dyke failed at Wilnis, about 30 km south-east of Amsterdam (van Bars, 2005). Like in Mayo and on the Shetland Islands, the summer of 2003 was the driest in 50 years in the

Netherlands. Failure of the dyke was attributed to a reduction in unit weight in the peat, which makes up the dyke, causing a reduction in vertical effective stress and hence instability.

Failure mechanism

The dry conditions in the summer prior to the slide would have resulted in drying and shrinkage of the blanket peat (Figure 4b). This caused new cracking, reactivation of old cracks and opening of peat fuel cuttings. These cracks were observed by local people in the area before the event and in undisturbed areas adjacent to the slides afterwards. Any increase in stability due to lowering of the water table is likely to have been offset by the reduction in unit weight of the peat by drying. This matter will be addressed in the stability analyses below.

During the intense rainfall, water would have rapidly percolated to the base of the peat, via the new and old cracks and the pipe network towards the top of the weathered rock. Here drainage would have been prevented locally by the “hard pan” and impermeable bedrock. It is likely then to have spread away from its entry points through the ground and the drainage conduits. Pore pressures in the weathered rock and basal peat would have increased significantly, reducing the effective stresses and the resistance to sliding.

It is also possible to speculate that repeated drying and wetting cycles caused shrinkage and swelling movements in the basal peat (Warburton et al. 2004). These movements could have been accommodated in the zone just above the “hard pan”. A plane of weakness could have been gradually developed. This phenomenon warrants further study

The sliding activity involved two distinct failure types (Tobin 2003, Duggan 2004):

- A. Shallow translational sliding of peat and weathered rock from the upper slope.
- B. Shallow rotational sliding of thin layers of peat material on the lower slope.

Type A failures are located on the upper slope and are the dominant failure type (Figures 3 and 4). Apparent failure scars on the lower slopes, for example behind the cemetery at Pollatomish, are primarily as a result of landslide debris passing over the existing ground surface. The ground in these areas did not fail, though it was scoured by the passage of debris and subsequent erosion by surface water. Once mobile the soil material mixed with rain and groundwater and behaved as a fluid. Down slope flow was mainly via the existing drainage channel and small topographical features.

Type B failures seem to have been triggered by the impact of mobile material from Type A failures in the upper slopes.

Description of slides

The authors have identified about 50 distinct failure zones, about 30 of which are of significant (of more than a few metres width) size. Some of these coalesce to form larger failure masses. Some typical failures are shown on Figure 3. Note that the transverse drainage ditch absorbed a significant fraction of the slide energy and protected the privately owned land down slope. Figure 4 shows a detail of a failure on the upper and middle slopes. Scar widths varied between a few metres to in excess of 150 m and involved volumes of material up to 20000 m³. The total volume of material involved is estimated to be in the region of 200,000 m³.

In general damage was relatively minor. Some structural damage was caused to a few buildings and a minor public road was blocked and damaged. Damage to the local graveyard was the cause of most distress to the local population.

Survey of area and slip surface

The majority of the failures were of Type A and, from observations of the head and side scars were located in the mineral soil above the “hard pan”, see Figure 5(a). This surface was extremely smooth and difficult to walk on. It exhibited striations or scratches in the down slope direction. Some orange colour staining of poorly developed “hard pan” can be observed below the failure surface. Within the weathered rock the failure surface was typically at about 200 mm depth and resembled weathered joints or discontinuities in a rock mass due to its “rusty” appearance and the presence of a seepage zone. Subsequent water flow may have locally exposed the surface of the intact bedrock, Figure 5(b).

The depth to the failure surface varied between 0.4 m and 1.0 m with an average of about 0.75 m. The inclination of the failure plane at these locations ranged between 22° and 58° with an average of about 37°. At these locations there was 0.3 m to 0.7 m of peat with an average of about 0.5 m.

Of the 50 individual failures, it is estimated that in about 11 of these the failure surface was located in the peat. These failures were of Type B, i.e. shallow rotational slides triggered by mobile sliding material. The failure plane comprises a thin polished layer (typically 3 to 5 mm) of stiff black to dark brown amorphous silt/clay located near the base of the peat, see Figure 6.

They were distributed approximately evenly over both the north and south sides of Dooncarton Mountain. At these locations both the peat thickness and the depth to the

failure surface varied between 0.3 m and 0.9 m. On average the failure surface occurred at 0.7 m and the peat was 0.75 m thick, i.e. failure occurred close to the base of the peat. Slope angles at these locations ranged between 12° and 35° with an average of about 22°.

In summary it seems that in general failures took place in the mineral soil on the steeper slopes. However where the peat was thicker the failure plane was located within this material and at these locations the slope was generally shallower.

Site Investigation

Block samples of the material in the failure surface were obtained by hand carving. These samples were approximately cubical with sides of 300 mm. They were sealed by successive layers of plastic film and tin foil, packed in wooden crates and transported to the laboratory. Little disturbance was evident in the samples and it was felt that they were of good quality.

Due to the sensitivity of the situation and the ongoing remedial work, it was not possible to obtain peat samples from the area of the slips. Instead some investigations of similar blanket peat from a slope about 4 km south west of Pollatomish were carried out. These tests were carried out adjacent to one another so as to minimise peat variability. They comprised in situ piezocone (CPTU) testing and vane strength testing (using the Geonor H10 130 mm x 65 mm vane). Although many authors have expressed reservations about vane testing of peat (e.g. Landva, 1980) they remain the most usual test for peat strength employed in Irish practice and therefore they were included in this study. Samples were obtained using the Swedish Geotechnical Institute (SGI) peat sampler, see Figure 7. This simple sampler has a sharp serrated edge and produces

samples of about 9 cm in diameter and 1 m in length. They are retained within a simple PVC drainage pipe and sealed with wax.

Laboratory testing

This comprised:

- Basic index properties (moisture content, bulk density, particle size distribution and plasticity) of peat and weathered rock.
- Organic content (by loss on ignition, LOI, at 440° C) and linear shrinkage of peat
- Small (60 mm square) shear box testing of weathered rock. Size considered satisfactory was 80% of the material was less than 1 mm in size.
- Triaxial testing of peat and weathered rock.

The triaxial test involved both isotropically consolidated (CIU) tests where the samples were consolidated to a low stress (≈ 5 kPa) to reflect the in situ conditions or simple unconsolidated undrained tests (UU) with pore pressure measurements.

Results of fieldwork and laboratory testing

Peat-basic properties

Basic index properties of the peat and a description based on the von Post scale are given on Figure 8. The studied site was underlain by about 1.8 m of H4 to H5 (somewhat to moderately humified) peat, over about 0.6 m of H6 (moderately humified) peat, which in turn overlies a thin 0.2 m layer of H7 to H8 (fairly well to well humified) material. Fibres and roots are still easily identifiable. They comprise a mixture of coarse and fine elements and are orientated and spaced randomly. The material shows little evidence of plasticity. Weathered rock was encountered at 2.7 m. Peat is notoriously heterogenous. However in this case it can be seen that the properties are relatively uniform, with its

moisture content varying between 800% and 1200% (average 1055%) and with an average bulk density of 1.03 Mg/m^3 , i.e. slightly denser than water. Based on the LOI values the material is almost completely organic with an average organic content of 97.9%. Linear shrinkage values reduce from close to 50% at 0.75 m to about 40% at 2 m depth, possibly reflecting the reduction in fibre content with depth. Moisture content at the shrinkage limit is about 10% confirming the large potential for shrinkage in the material.

The nature of the peat and the contrast between the peat and the underlying weathered rock can be seen clearly in the plot of the CPTU data on Figure 9. Initially the CPTU end resistance (q_c) is relatively high due to the fibrous nature of the upper peat layers. It then gradually reduces to a minimum at about 1.5 m, before increasing slightly again. The interface with the underlying weathered rock can be clearly seen in the q_c response. CPTU friction ratio (FR) decreases reasonably uniformly with depth in the peat reflecting the reducing fibre content (increasing H number). The generated pore pressure (u_2) is sensitive to the rate of cone penetration. Here the standard rate of 2 cm / s was used and u_2 generally greater than hydrostatic, particularly in the upper more fibrous material, until close to the weathered rock surface.

Peat -undrained shear strength

Undrained shear strength (s_u) of the peat was determined in situ using the field vane and in the laboratory by means of triaxial tests. These comprised both isotropically consolidated undrained (CIU) tests and unconsolidated undrained tests (UU). For the former tests the specimens were consolidated to a stress of about 6 kPa to reflect the

approximate mean in situ effective stress. Samples were 5.4 cm in diameter and 10 cm high. Shearing was at 18% per day.

Although the fibres will influence the shearing action in different ways for the vane and triaxial tests, there is very good agreement between the values from the 3 sets of tests, as can be seen on Figure 10, with s_u increasing from about 5 kPa at 1.0 m depth to 8 kPa near the base of the stratum. Undrained shear strength can also be determined from CPTU tests. Rodgers (1992) found that there was good correlation between effective cone resistance $q_t - u_2$ (q_t is cone resistance q_c corrected for pore pressure effects after Lunne et al. 1997) and vane shear strength for various (including organic) soils. Using the expression $s_u = (q_t - u_2) / N_{ke}$ and assuming $N_{ke} = 9$, it can be seen that s_u derived from the CPTU compares very well with those values obtained from the other techniques.

Field vane sensitivity (S_t) values for the peat are also shown on Figure 10 and it can be seen that S_t increase from about 8 to 18 with depth. These are relatively high values and are consistent with the near liquid state of the peat following remoulding in particular during the Type B slides.

Peat effective stress strength parameters

As the failures in this case were probably due to higher than normal water pressures, any stability analysis of the slides must be undertaken using effective stress strength parameters. Due to its fibre content and the large volumetric strains which occur during consolidation, very high values of effective friction angle are often measured in peat. For example Den Haan et al. (1995) report ϕ' values of between 32° and 58° for Dutch organic soils, with ϕ' increasing with decreasing density. In this study efforts were made to accurately characterise the peat strength by consolidating the samples to the best

estimate of in situ stress as described above and by attempting to separate fibrous and frictional effects.

Stress paths (in $s' = \sigma'_1 + \sigma'_3 / 2$, $t' = \sigma'_1 - \sigma'_3 / 2$ space) for three of the tests are shown on Figure 11. It can be seen that the behaviour of the peat resembles that of a normally consolidated clay and that its parameters can be defined as being in the range $c' = 3$ kPa and $\phi' = 35^\circ$ to $c' = 5$ kPa and $\phi' = 35^\circ$.

In fibrous peat high ϕ' values are due to the reinforcing effect of the predominantly horizontally orientated fibres. Landva and Rochelle (1983) attempted to account for these effects by comparing triaxial test results with ring shear tests where the fibre effect is not mobilized due to the large deformation. From large ring shear tests on natural fibrous peat, Landva and Rochelle (1983) found values of approximately $c' = 3$ kPa and $\phi' = 32^\circ$. They combined this envelope with earlier triaxial test results reported in the literature, (by Hanrahan 1954 and Gautschi 1965), where horizontal effective stress reached zero at failure. Keeping σ'_1 constant they increased σ'_3 to bring the stress state on the ring shear envelope. The increase in σ'_3 is then the apparent increase in lateral resistance due to the effect of the fibres.

The effect of the fibres can then be expressed as $\sigma' \tan \alpha$ and as can be seen on Figure 12, this effect is consistent being highest for H1 peat and more or less zero for H9 peat. The data from this site is consistent with the older results and there would seem to be considerable promise in using this scheme. From this approach then the fibres account for approximately 4° of the frictional resistance of the material and combined with Landva and Rochelle's ring shear data gives an overall strength of $c' = 3$ kPa and $\phi' = 35^\circ$, which is consistent with the data from the stress path plots on Figure 11. It is recognised that

combining data from different parts of the world in a material which is highly variable is questionable. However the objective here is to obtain parameters for analyses which will attempt to identify trends and not for a detailed analysis of one particular slide.

Weathered rock – basic parameters

A block sample of the weathered rock, from the failure zone of a typical Type A slide on Dooncarton Mountain, was sub-divided into four portions for shear box and triaxial testing as shown on Figure 13. A grading curve for the material is also shown in Figure 13. In engineering terms the material can be described as medium dense dark brown peaty sand. Basic index properties are very variable with no obvious correlation to the zone of most intense shearing. Moisture content varies between 43% and 88% with an average of 63.5%. Similarly bulk density varies between 1.4 Mg/m^3 and 1.65 Mg/m^3 with an average of 1.55 Mg/m^3 .

These relatively high moisture content and low density values are suggestive of some leaching and washing out of the material and mixing with peat. This leaching process was studied by O'Dubháin (1978) for similar soils in the east of Ireland. The upper, less humified, peat is the main source of the acid leaching which combines with iron and aluminium in the upper zones of the weathered rock. These organo-mineral complexes are then leached lower into the material where they accumulate to give the organic character to these horizons, as noted above.

Weathered rock – behaviour in shear and effective stress strength parameters

Results of the small shear box tests for the weathered rock are shown on Figure 14. Normal stresses were 5.4 kPa, 10.4 kPa and 21.5 kPa, these values being as close as were possible to the in situ stress conditions. Following initial shearing the box was reversed

and the sample was re-sheared in order to observe the large displacement shear strength.

From the plots it can be seen that:

- Material strength increases with increasing normal stress.
- Behaviour is ductile with little difference between peak and large displacement strength.
- Peak and large displacement strengths correspond approximately to $c' = 0$ kPa and $\phi' = 45^\circ$.

Vertical movement of the shear box was also measured during the tests and only very small movements were detected indicating the state of the material was close to the critical void ratio. The ductility of the material is an important factor as far as sliding is concerned as it indicates that rapid sudden failure without warning is unlikely.

Results of a CIU triaxial test on the weathered rock material are shown on Figure 15. A consolidation stress of 20 kPa was used, this being the resolution of the particular device. The ductility of the material can be seen in the stress / strain curve with a gradual build up to peak. As can be seen from the stress path plot the material behaves in a dilatant manner, which is characteristic of sands. In this case the test suggests the strength parameters correspond to $c' = 0$ kPa and $\phi' = 38^\circ$.

Analysis of slide

In this section it is not intended to analyse one specific slide. Indeed due to the complexity and variability of the peat material and of the drainage conditions, it is questionable whether it is possible to accurately model such slides. Instead a more general analysis will be carried out, with an idealised profile, to identify patterns and trends.

Input parameters

Based on the laboratory test data presented above, the parameters summarised on Table 1 were adopted for the stability calculations. For the case of the weathered rock more emphasis was placed on the results of the triaxial test as it is well known that small shear box tests tend to overestimate strength (e.g. Potts et al., 1987) and also the box used was relatively small.

Limit state equilibrium analysis by method of slices

In order to calibrate the relatively simple infinite slope analysis, some calculations were carried out using the method of slices by means of the software OASYS SLOPE, see Figure 16. The example shown is for a 40° slope with 1.2 m thickness of peat and with the failure surface located within the base of the peat. For this example groundwater is assumed to be at the base of the peat. As can be seen the failed mass was subdivided into 10 slices and the factor of safety was calculated using Janbu's method of variably inclined inter-slice forces (Janbu 1957). A factor of safety (FoS) of 1.43 was calculated. For the same conditions, except with a failure surface located in the weathered rock, the factor of safety was calculated to be 0.87, i.e. tendency is for failure to occur in weathered rock on steep slopes, as was observed in practice.

Simple infinite slope analysis

The relatively simple infinite slope analysis is suited to long downslope lengths where the mode of failure is by translational sliding. For effective stress analysis, and assuming steady seepage of groundwater parallel to ground level, according to Haefli (1948) and subsequently Skempton and DeLory (1957):

$$FoS = \frac{c'}{\gamma z \cos \beta \sin \beta} + \frac{(\gamma - \gamma_w m) \tan \phi'}{\gamma \tan \beta} \quad (1)$$

where:

γ = bulk unit weight of soil

γ_w = bulk unit weight of water

β = slope angle on base of sliding

m = depth to groundwater measured upwards from slip surface

Some output for infinite slope analyses with various peat thickness and depth to the water table and with the failure surface located in the peat are shown on Figures 17. Water pressures in the weathered rock correspond to those at the base of the peat. Slope angles were assumed to equal 20° and 40°, which were in the range of those typically observed in the field. It can be seen that:

- Thin layers of peat can be stable on steep slopes.
- For peat thickness of 1.2 m or greater and high m values, the slope becomes unstable.
- FoS decreases with increasing layer thickness.
- FoS decreases with increasing slope angle.
- Increasing water pressure (m) causes significant reduction in FoS.
- For $m = 0$, peat thickness = 1.2 m and slope angle = 40°, FoS = 1.34. This compares to 1.43 from the more sophisticated method of slices.

Similar output is shown on Figure 18, except in this case a comparison is made between failure in the weathered rock and in the peat for the case of a 40° slope. In this case it can be seen that:

- There is a greater potential for failure in the weathered rock except where the peat becomes thick.

- For $m = 0$, and failure in weathered rock, $FoS = 1.00$. This compares to 0.87 from the more sophisticated method of slices.

Sensitivity analyses

Computed factors of safety will, in this case, be sensitive to several input values. In the authors view the two most uncertain critical parameters are:

- Bulk unit weight of dried / partially dried peat
- Shear strength parameters of peat, in particular c'

No specific measurements of the unit weight of dried peat have been carried out for Pollatomish. Data for the nearby peat sampling site, shown on Figure 8, which was obtained close to the time of the failure at Pollatomish suggests very little influence of drying on the bulk density. Van Baars (2005) has determined an average bulk unit weight of 5 kN/m^3 for the unsaturated peat at the Wilnis embankment and this value is used here for illustration purposes. Taking the case of the 40° slope with peat thickness 1.2 m, as shown on Figure 19, FoS can be seen to be extremely sensitive to this parameter for all cases except the dry slope. By comparing the slope of the plots, it seems the influence of the decrease in effective stress caused by the reduction in peat unit weight is not counterbalanced by the corresponding reduction in water pressure.

Also it can be seen that FoS is, as expected, reduced significantly by decreasing the peat c' value from 3 kPa to 1 kPa. These sensitivity calculations give a focus for future studies.

Discussion of stability analysis

An accurate and detailed stability analyses of slides, such as those which occurred at Pollatomish, would be extremely difficult if not impossible to carry out. It is not clear

how the natural material variability of the peat, its time dependent parameters and the influence of fibres could be reliably modelled. Drainage conditions, particularly with respect to the local pipes within the peat and weathered rock, are also very complex. In this paper a relatively simple analysis is adopted in order to model the general situation observed at Pollatomish. The sensitivity analysis conforms that the choice of input parameters, particularly those of the bulk unit weight and strength of the peat, need to be carefully chosen.

Conclusions

The features of many of the landslides noted in the field at Pollatomish were readily reproduced by the relatively simple analysis described. These features included:

- Thin layers of peat can remain stable on steep slopes.
- An increase in pore water pressure drastically reduces the margin of safety. These high pore pressures occurred on site due to the combination of the intense rainfall and the prevailing drainage conditions. Dry weather in the preceding summer had created preferential flow paths to the base in the peat through shrinkage cracks.
- The analysis predicts that the most likely failure zone is the surface of the weathered rock. In the field most of the failures in the steep upper slopes occurred in this material.
- The analysis also suggested that failure could occur in the peat when its thickness was appreciable. This was also observed on site.

This study suggests that a relatively simple method of analysis can be used to assess the risk of future landslides in similar conditions. However, even for this simple analysis, it is first necessary to carefully characterise both the peat and underlying mineral soils

and to have a good knowledge of the prevailing groundwater and drainage conditions. There seems to be some promise in the use of the technique of Landva and Rochelle (1983) for understanding the effect of fibres on the shearing behaviour of peat. Some sensitivity analyses have provided a focus for future studies into the critical peat parameters.

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Table 1. Input parameters for stability analyses

Material	Bulk density (Mg/m ³)	ϕ' (Deg.)	c' (kPa)
Peat	1.03	35	3
Weathered rock	1.55	40	0

List of figures

Fig. No.	Title	Reference
1	Site location / layout	D(AE)/Pollatomish/Ireland.jpg+mapmayo.grf
2	Schematic section through slope	D(AE)/Pollatomish/slopeprofile.xls+.grf
3	General view of failure zones	Picture
4	Detail of failure on upper and middle slopes and shrinkage crack in peat	Picture
5	Detail of Type A slip surface immediately after slide and 18 months later	Picture
6	Detail of Type B slip surface showing nodular hard pan	Picture
7	SGI peat sampler	Picture: D(AB)/SGISampler1a.jpg
8	Index properties and classification of peat	Laptop/labtests/corrib/basics.xls+.grf
9	CPTU test results in peat	Laptop/labtests/corrib/cptuwest.xls+.grf
10	Undrained shear strength of peat	Laptop/labtests/corrib/sucpt.xls+.grf+basics.xls
11	Effective shear strength of peat	Laptop/labtests/corrib/stresspaths.xls+smoothstresspaths.grf
12	Fire effects	D(AA)/Polishpeat/fibreffect.xls+.grf
13	Basic index properties of weathered rock	D(AE)/Pollatomish/psd.grf+blocksample.grf
14	Shear box tests in weathered rock	D(AE)/Pollatomish/peakres.grf+smallshearbox.grf
15	CIU triaxial test in weathered rock	D(AE)/Pollatomish/triaxial.grf
16	SLOPE analysis	D(AE)/Pollatomish/Pol1.sld
17	Infinite slope analysis – varying peat thickness	D(AE)/Pollatomish/Infinitepeat.grf+xls+infinite slope.xls
18	Infinite slope analysis – comparison of failure in peat and weathered rock	D(AE)/Pollatomish/Infinitepeat.xls+infinite slope.xls+infinite wrk.grf
19	Infinite slope analysis – sensitivity to input parameters	D(AE)/Pollatomish/Sensitivity.grf+Infinitepeat.xls

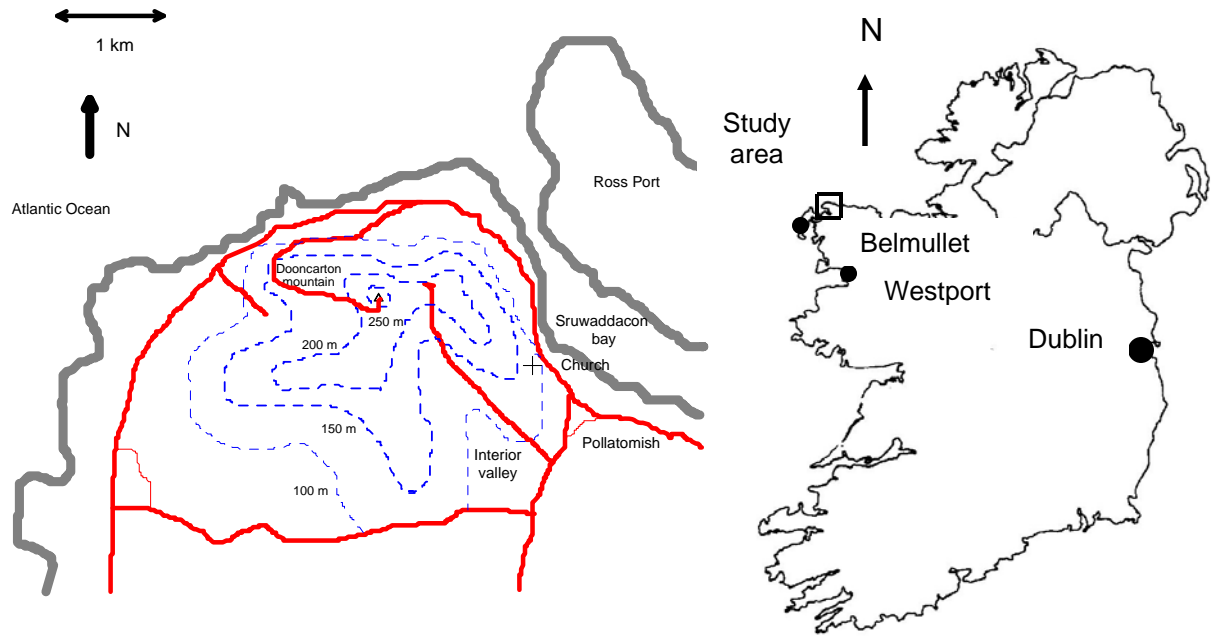


Figure 1. Site location and layout. Red lines indicate roads. Blue lines are contours. Grey line is coast. Failures occurred in various locations but were mostly concentrated on north and east sides of Dooncarton Mountain and in interior valley.

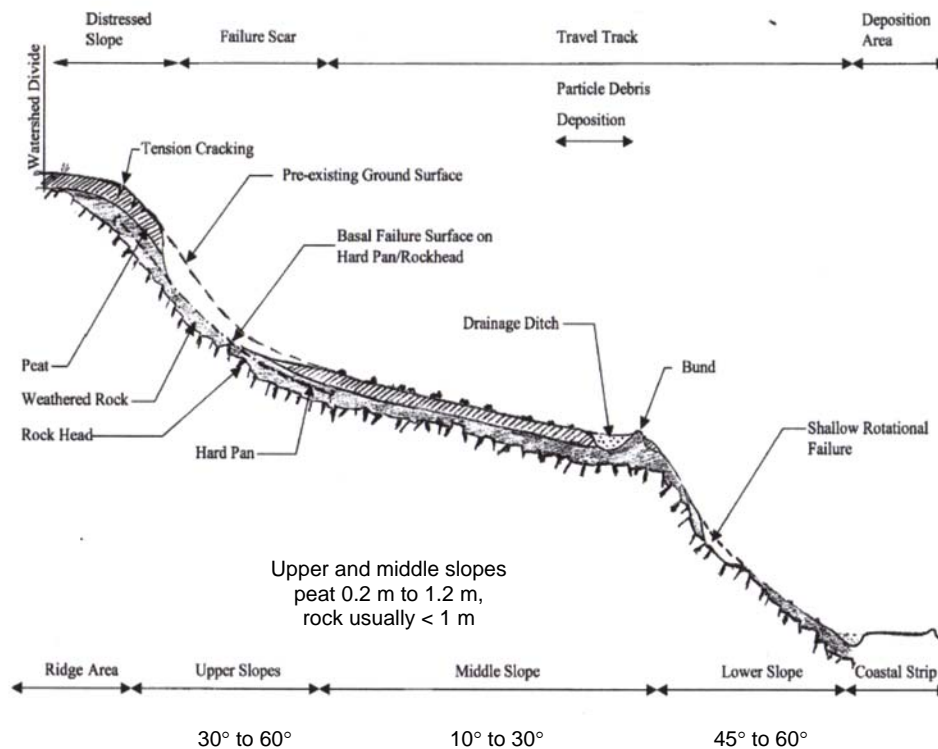


Figure 2. Schematic section through slope (adapted from Tobin 2003)



Figure 3. General view of slips on north side of Dooncarton Mountain facing towards the west. Type A failures mostly in upper slopes. Type B failure near middle house. Note movement arrested by drainage ditch at base of middle slope. Debris flow into lower slope cause by water flow.



Figure 4. (a) Detail of failures on upper and middle slopes exposing smooth weathered rock surface; (b) detail of shrinkage crack in peat surface in undisturbed area adjacent to slide.



Figure 5. Type A slide slip surface (a) just after slide showing smooth surface in weathered rock and downslope striations and (b) approximately 18 months later where water flow has cleaned surface and locally exposed intact bedrock



Figure 6. Slip surface in Type B slides: failure plane comprises a thin polished layer (typically 3 to 5 mm) of stiff black to dark brown amorphous silt/clay located near the base of the peat. This layer overlies weathered rock with local nodular "hard pan".



Figure 7. SGI peat sampler

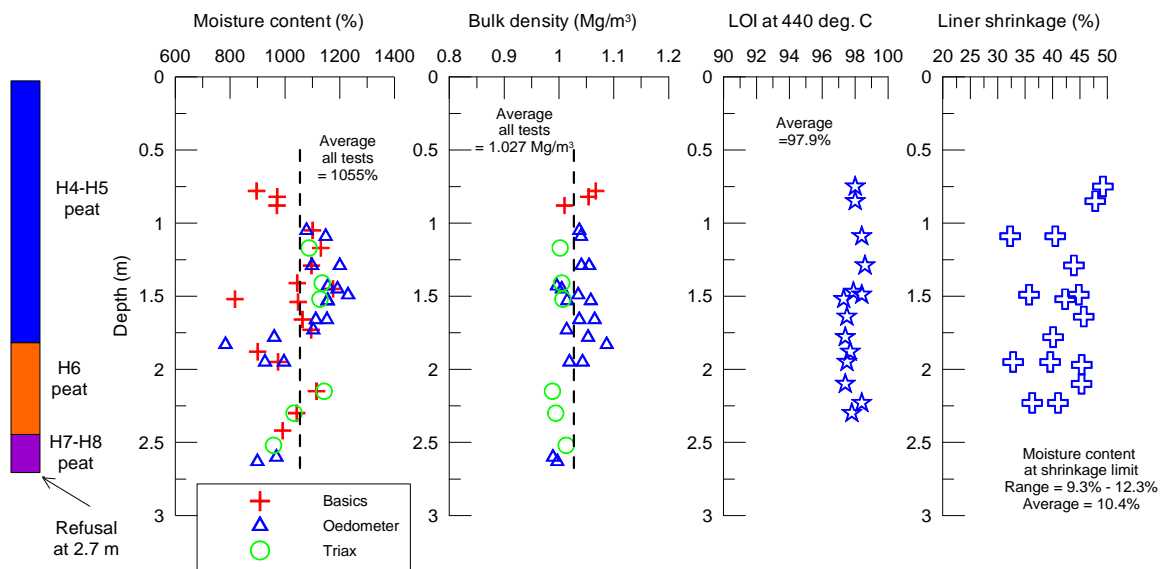


Figure 8. Index properties and classification of peat

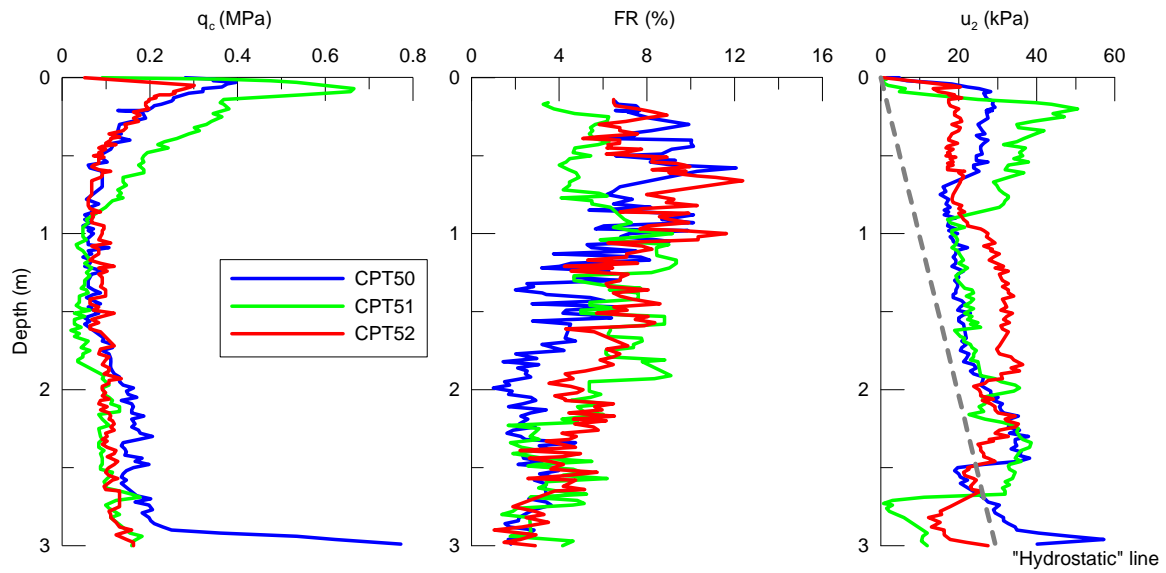


Figure 9. CPTU test results

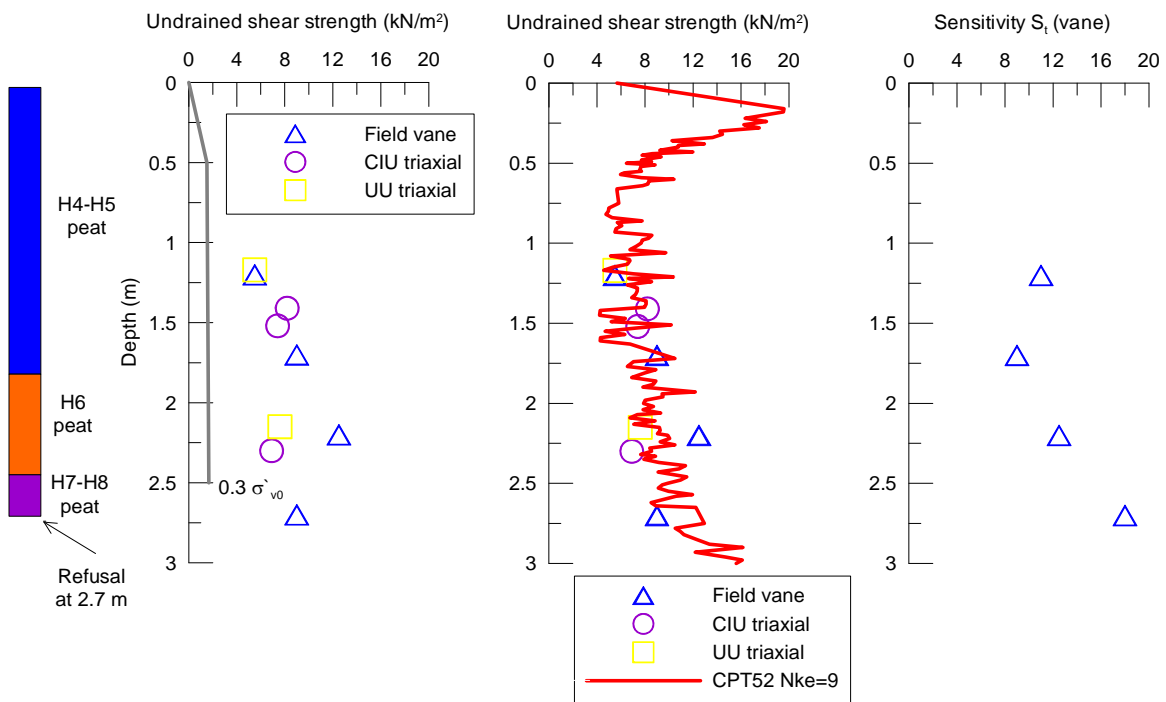


Figure 10. Undrained shear strength

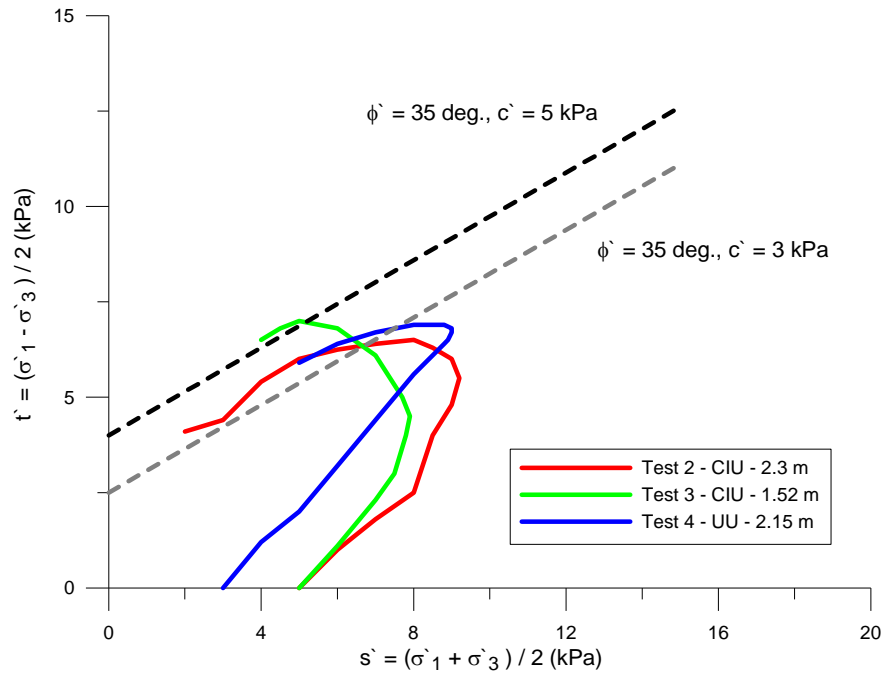


Figure 11. Effective stress strength parameters / stress paths

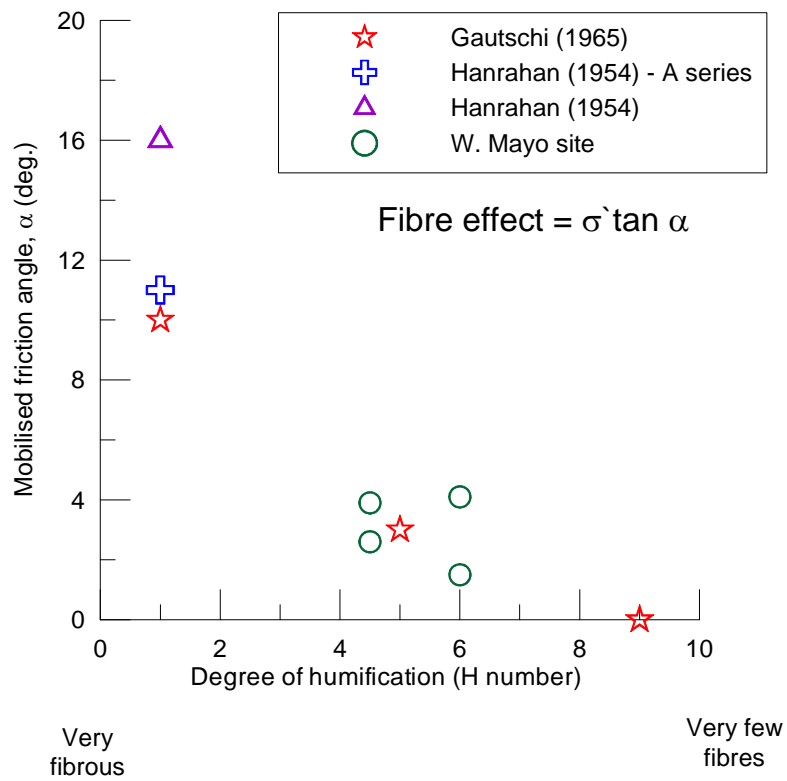


Figure 12. Effect of fibres on effective strength parameters

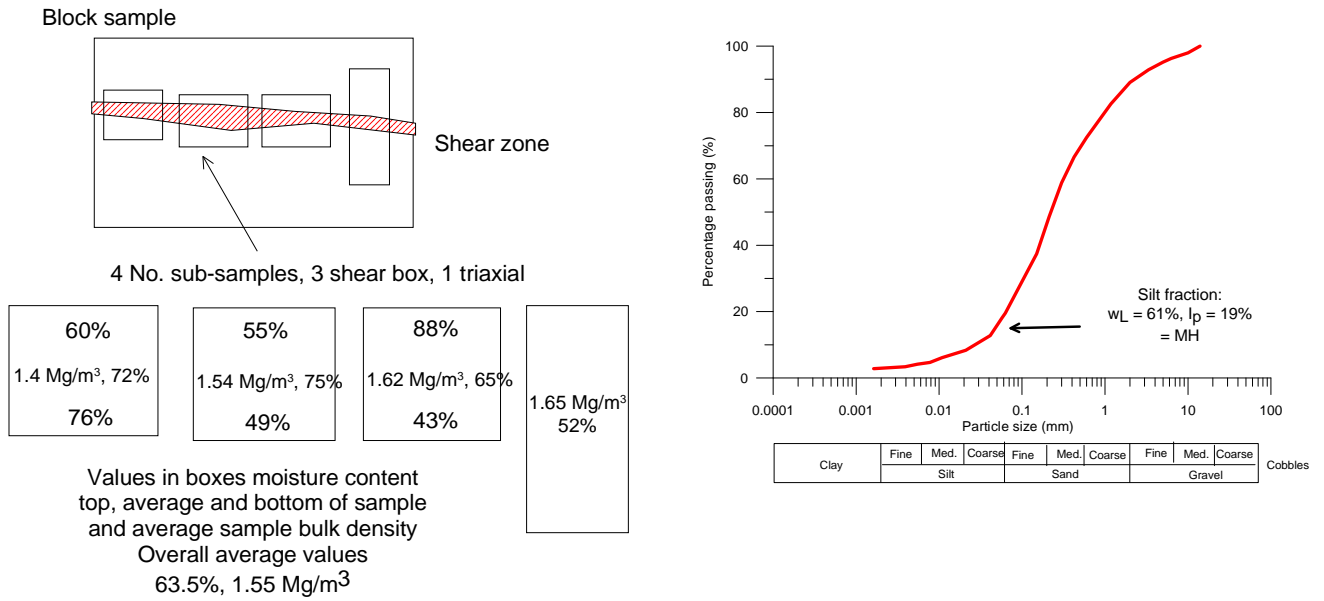


Figure 13. Basic index properties of weathered rock in failure zone.

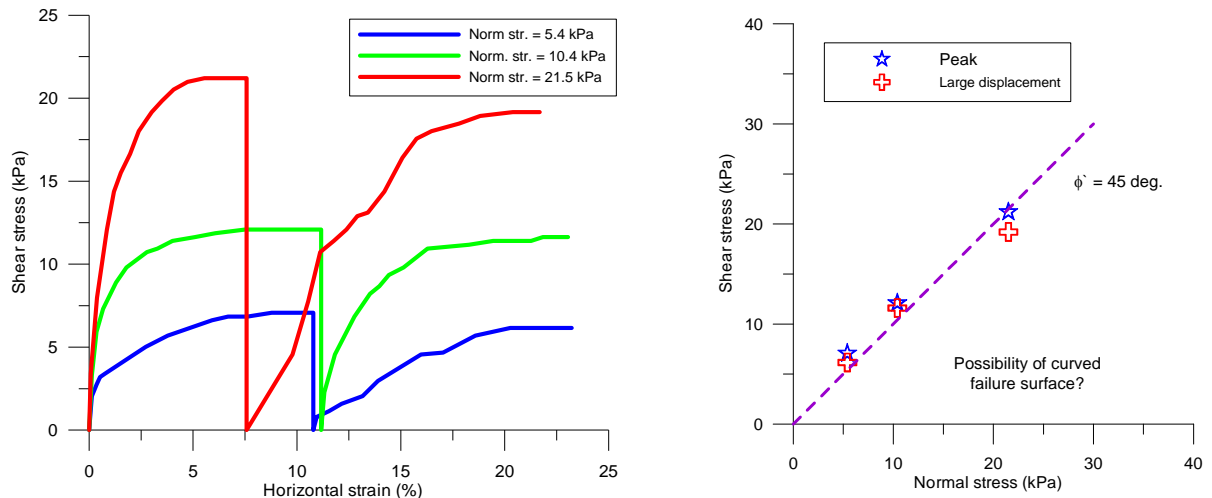


Figure 14. Shear box tests in weathered rock

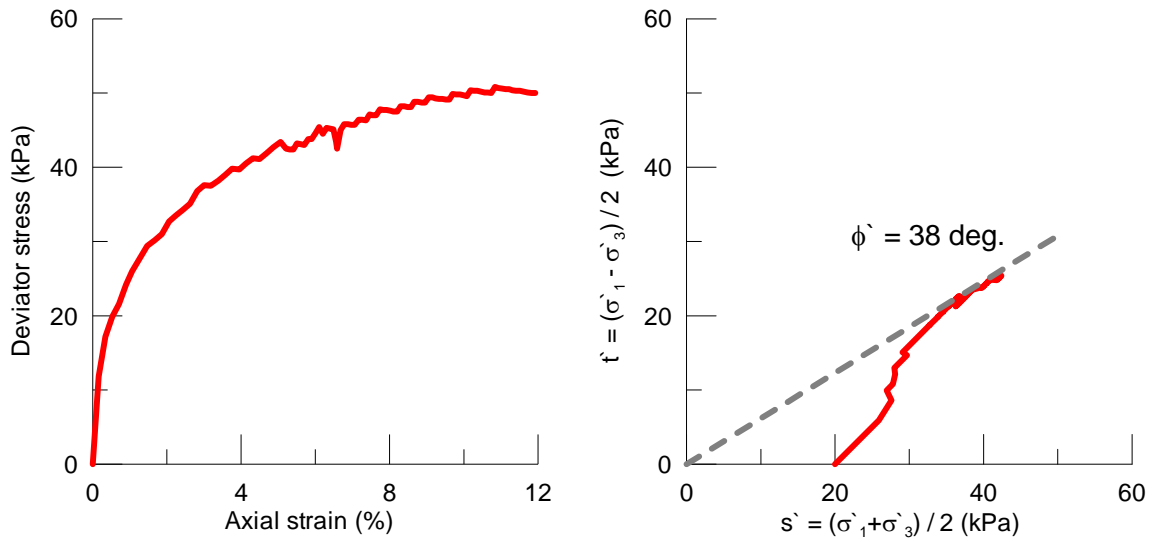


Figure 15. CIU triaxial test in weathered rock

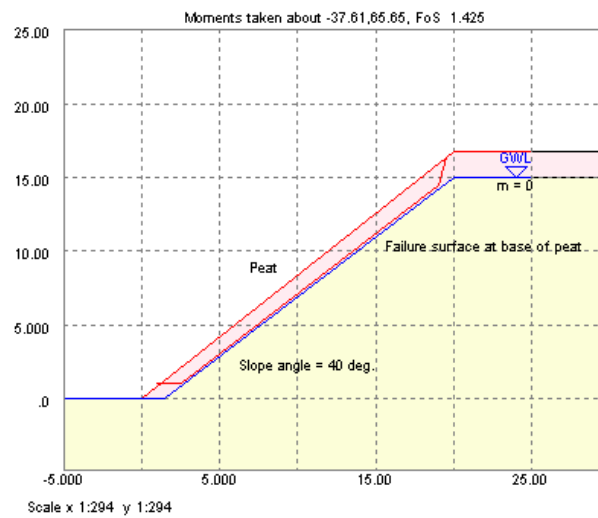


Figure 16. Output from OASYS-SLOPE. (Slope angle 40°, peat thickness 1.2 m, water at base of peat, failure surface near base of peat)

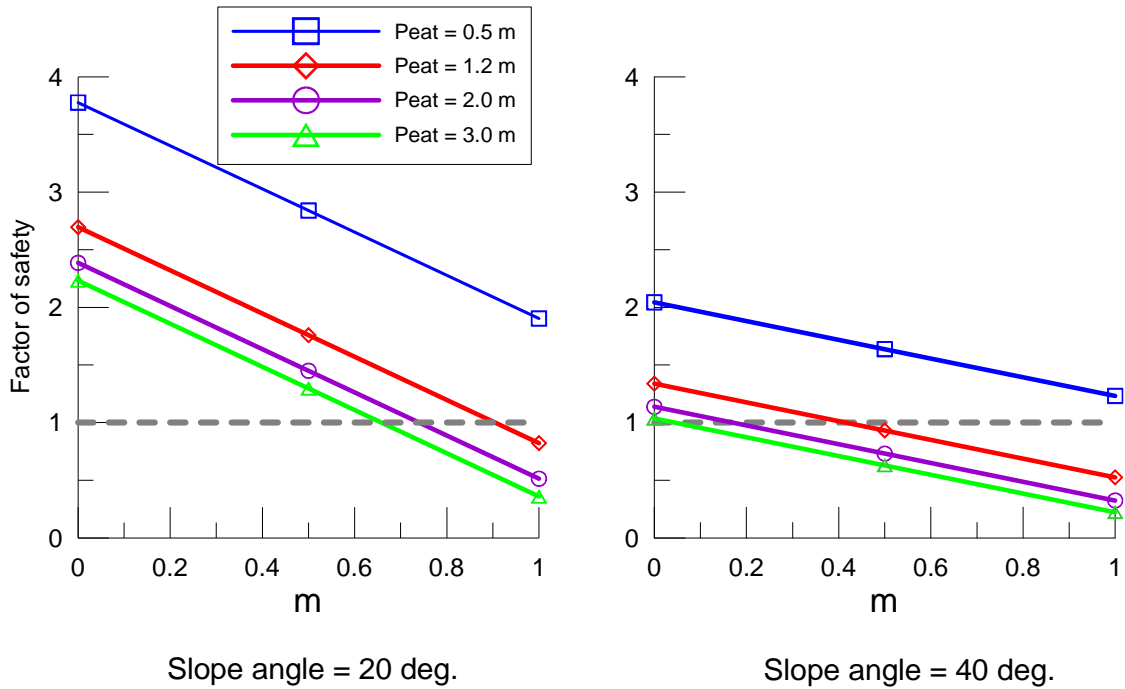


Figure 17. Infinite slope analysis – failure in peat. m is depth of water measured upwards from failure surface

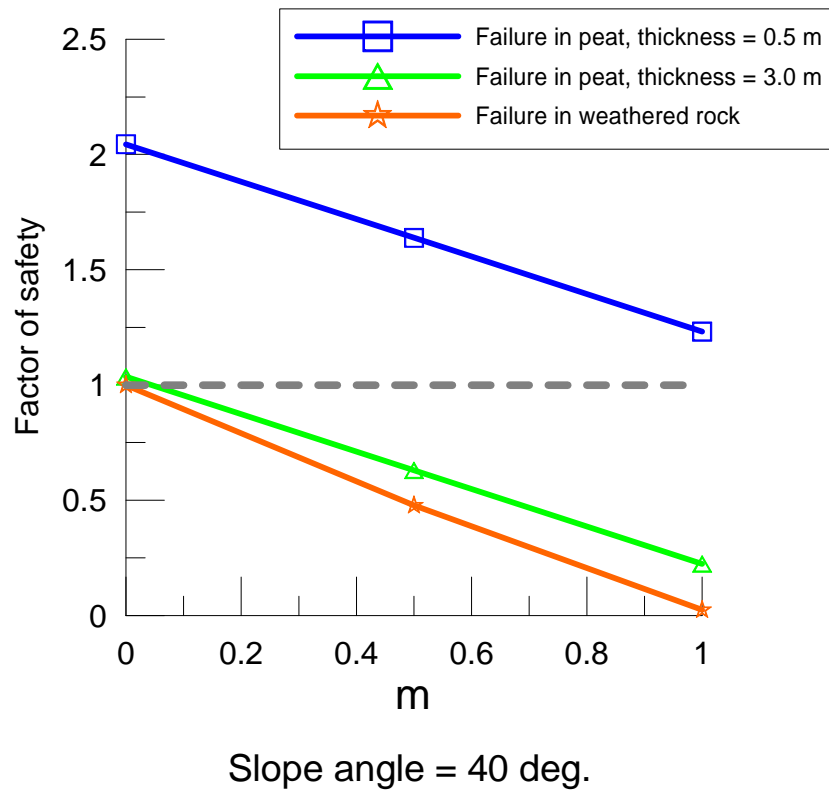


Figure 18. Infinite slope analysis – comparison of failure in peat and weathered rock

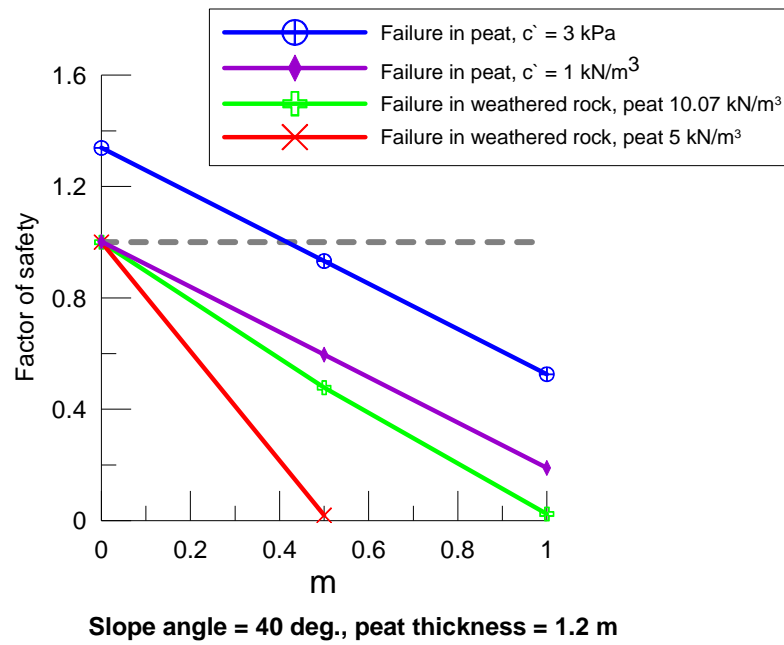


Figure 19. Infinite slope analysis – sensitivity to input parameters of peat