Geotechnical stability of peat dams and embankments

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ABSTRACT: Peat material has been successfully used in the construction of canal embankments, dykes and more recently dams erected to engineer the regeneration of bogs. However, a number of geotechnical failures have occurred that have caused extensive damage. The peat embankments and dams generally fail by the lateral displacements of large intact blocks of the peat material as opposed to traditional circular slip type failures. Horizontal and vertical stability analyses were carried out for three case studies considering the retained depth of water and the effects of reductions in the embankment self weight following partial drying of the crest material.

1 INTRODUCTION

Peat material has been successfully used in the construction of canal embankments, dykes and dams. For example, the Grand Canal and the Royal Canal traverse the Bog of Allen, a collection of raised bogs in the Irish Midlands. Large peat embankments (within which the lines of the canals were later excavated) have been constructed on the bog surface (Figure 1).

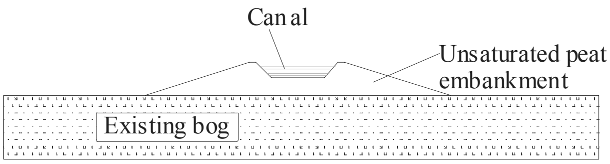

Figure 1 - Canal embankment of peat material.

In the Netherlands, a large area of the west of the country is reclaimed polder land that is susceptible to flooding. The surface water drains into the nearby rivers and the sea via a network of peat dykes.

More recently, peat dams have been used to engineer the conservation of bog ecosystems that have been affected by drainage schemes for land improvement or turf harvesting. The level of the bog surface subsides in response to the lowering of the naturally high groundwater table as a result of consolidation and air-drying of the surface peat layer. Stunted plant growth reduces the ability of the bog to naturally regenerate. Dams constructed of locally available peat material have been successfully used to re-establish the original high groundwater level. The dams are constructed along the bog perimeter to pond surface water thereby re-saturating the surface peat layer and encouraging its recolonisation by indigenous plants.

2 GEOTECHNICAL STABILITY

Three case studies of geotechnical instability are presented for further study, namely:

- Peat embankment failure at Edenderry, County Offaly;
- Peat dyke failure at Wilnis, The Netherlands;
- Peat dam failure at Raheenmore bog, County Offaly.

The peat embankment failure occurred along the Grand Canal near Edenderry in 1989. The embankment, about 115-m in width at the base and 10 m in height, had been constructed on the bog surface by compacting partially dried peat material that had been placed in 0.5-m lifts. A channel 13–16 m in width was later excavated in the top of the embankment to form the line of the canal. The embankment failure (Figure 2) involved the lateral movement of up to 60 m of an intact block of peat material about 225-m by 105-m in plan area (Pigott et al. 1992).

Figure 2 - Canal embankment failure at Edenderry (Pigott et al. 1992).

The slip plane was inclined at 2.5^{o} to the horizontal and extended into the bog beneath the embankment. An estimated 200,000 m^3 of displaced peat material and 135,000 m^3 of flood water caused extensive damage to about 6 ha of the neighbouring land and temporary flooding to another 18 ha.

The peat dyke failure (Figure 3) at Wilnis, located about 30 km southeast of Amsterdam, occurred in August 2003 following a particularly dry summer. An intact block of peat material about 60-m in length was displaced laterally by about 10 m. Although repair works were started without delay, more than 600 neighbouring houses were flooded.

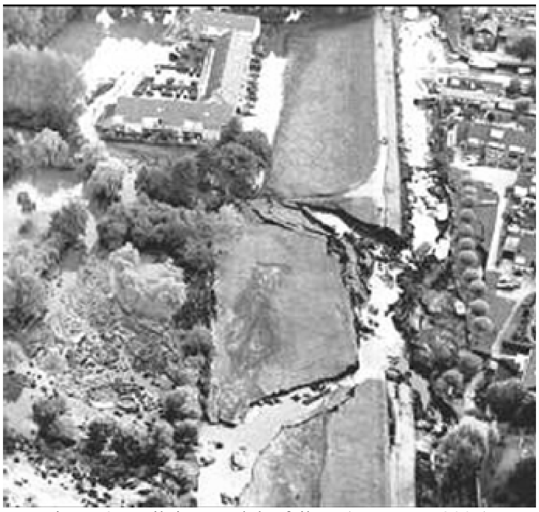

Figure 3 - Wilnis peat dyke failure (van Baars 2005).

The system of peat dams at Raheenmore bog, County Offaly, had been constructed as a conservation measure to encourage the natural regeneration of sections near the cutaway bog perimeter. A local slip failure along one of the dams occurred in December 1998 with the crest of the dam being displaced laterally by up to 1.5 m (Figure 4). An area of about 0.36 ha was affected by the resulting discharge of water and peat debris (Bennett, 1998).

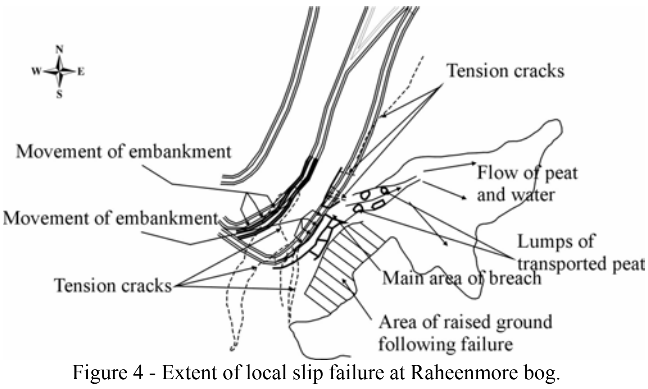

Figure 4 - Extent of local slip failure at Raheenmore bog.

A common failure mechanism, namely the lateral displacements of large, relatively intact blocks of peat material, occurred in the three case studies presented. The effects of drying out of the peat material at the crest of the embankments or dams during periods of extended dry weather combined with an increase in the active hydrostatic pressures during subsequent heavy rainfall events appeared to be the principal contributing factors. The slip planes were comparatively flat and were generally located along the base of the embankment or dam.

3 SLOPE STABILITY METHOD OF VAN BAARS

van Baars (2005) carried out a detailed study of the Wilnis failure taking into account the effects of drying out of the peat material near the crest of the dyke during the dry summer that preceded the failure and the effect of an increase in the active hydrostatic pressures arising following an intense rainfall event.

The method used by van Baars to back-analyse the failure considers horizontal and vertical stability allowing for the self weight of the embankment, the frictional resistance between the embankment and the foundation material, the pore water pressures and the active hydrostatic pressures. Consolidated undrained triaxial tests were carried out by van Baars on peat specimens as well as 12 direct shear tests under applied normal stresses of 10, 25 and 40 kN/m^2. The direct shear tests were deemed more appropriate for testing the cross-anisotropic peat material. c' = 2.5 kN/m^2 and ϕ' = 25^o were determined for the peat and used in the stability analysis. The embankment, when analysed using horizontal and vertical stability calculations, was found to be unstable (Table 1).

Water level	Peat at crest	FOS
Normal	Saturated	1.25
Normal	Unsaturated following dry weather	0.93
At crest	Saturated	0.94
At crest	Unsaturated following dry weather	0.70

Table 1 - Factor of safety (FOS) for different scenarios, Wilnis.

4 STABILITY BACK-ANALYSIS

The canal embankment failure at Edenderry and the conservation dam failure at Raheenmore bog are back-analysed in this paper using the van Baars method. At Edenderry, the crest of the embankment may have dried out during extended periods of dry weather resulting in low degrees of saturation. A value of 5.0 kN/m^3 was assigned to the bulk unit weight of the dried out peat material. The effects of the increase in the active horizontal pressure acting behind the embankment during intense rainfall events were also considered in the analysis. At Edenderry, the design water depth in the canal was 1.8 m. However, a water level of about 3.0 m was indicated in the embankment cross-section prior to the breach (Pigott et al. 1992) indicating that the water level had almost reached the crest of the embankment. Tension cracks were located near the toe of the embankment prior to the failure. It is therefore likely that the dried out crest material and the increased active hydrostatic pressures following an intense rainfall event produced the Edenderry failure.

Figure 5 shows the forces acting on the failed block of peat material. It was assumed that no passive pressures were acting on the right hand side of the failed block because of the series of deep tension cracks at the embankment toe that were filled with water. Uplift pressures would still be expected under the embankment.

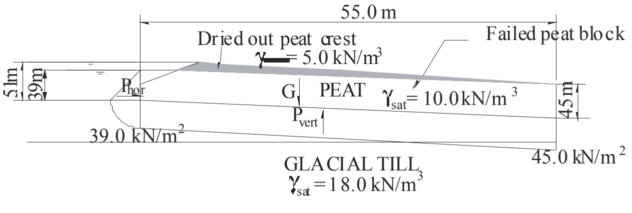

Figure 5 - Forces acting on failed block of peat, Edenderry.

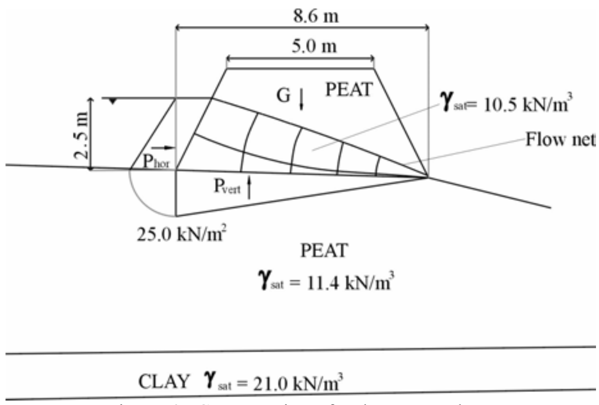

Figure 6 - Cross section of Raheenmore dam.

The total cross-sectional area of the failed embankment (I_{tot}) was 264 m^2. The self weight of the embankment if the peat material had been fully saturated over the entire cross section ($G_{sat} = I_{tot}$ x γ_{sat}) would have equalled 2640 kN/m run. The self weight of the embankment if the crest zone had been partially dried ($G_{unsat} = G_{sat} - I_{unsat}$ x $(\gamma_{sat} - \gamma_{dry})$) would have reduced to 2475 kN/m run. The hydrostatic uplift beneath the embankment ($P_{vert} = 0.5.(p_1+p_2).l$), where p is the water pressure underneath the embankment and l = the length of the failure plane, was 2310 kN/m run. The hydrostatic force ($P_{hor} = 0.5$ x γ_w x h^2), where h = 3.9 m, acting behind the embankment when the water level was at the design level in the embankment was 76 kN/m run. If the water level had risen to near the crest as indicated by Pigott et al. (1992) the hydrostatic force would have been 130 kN/m run. The limiting shear resistance mobilised over the base of the embankment was determined using c' = 0 and a ϕ' of 31^o (taken as a conservative estimate of ϕ' from previous shear strength testing on peat) in the equation: $F_{max} = l.c' + \tan\phi' (G - P_{vert})$. The factor of safety on slope instability (FOS) was calculated as F_{max}/P_{hor}, with failure occurring for FOS < 1.0. Table 2 shows the factors of safety calculated for the different scenarios.

Water level	Peat at crest	FOS
Normal	Saturated	2.61
Normal	Unsaturated following dry weather	1.30
At crest	Saturated	1.52
At crest	Unsaturated following dry weather	0.76

Table 2 - Factors of safety for different scenarios, Edenderry.

As a conservative value of ϕ' = 31^o was used in the preceding calculations, a sensitivity analysis was carried out the determine what value of ϕ' would result in FOS = 1.0 for the worst case scenario. A value of ϕ' = 38^o resulted in FOS = 0.99 for the case with the higher water level and dried out peat material at the crest.

Figure 6 shows a cross section of the outer conservation dam at Raheenmore, close to the failure location. The maximum depth of water retained by the dam was limited to 2.5 m by an overflow pipe that discharged down the protected slope of the dam. The total cross-sectional area of the dam (I_{tot}) was 24 m^2. If the peat material had been fully saturated over the full cross section, the self weight of the dam ($G_{sat} = I_{tot}$ x γ_{sat}) would have equalled 240 kN/m run.

The flow net in Figure 6 shows the general flow path through the dam and the zone above which partial drying out of the peat material may have occurred during extended periods of dry weather. From the bulk density tests and from values determined at other sites, a value of 3 kN/m^3 was assigned for the dried out peat material. If this zone had dried out, the self weight of the dam ($G_{unsat} = G_{sat} - I_{unsat}$ x $(\gamma_{sat} - \gamma_{dry})$) would have been reduced to 136 kN/m run. The hydrostatic uplift beneath the dam ($P_{vert} = 0.5$ x p x l) was 108 kN/m run. The hydrostatic force ($P_{hor} = 0.5$ x γ_w x h^2) acting behind the dam with the depth of water at the overflow level would have equalled 31 kN/m run. The limiting shear resistance mobilised over the base of the peat dam was calculated using the laboratory-measured c' = 0 and ϕ' = 38^o values in the equation: $F_{max} = l.c' + \tan\phi' (G - P_{vert})$.

Shearbox and ring shear tests were carried out in the laboratory in accordance with BS 1377 (1990) on the peat material in the dam and the bog. The consolidated-drained shearbox specimens were 101.6 mm in diameter and 20.0 mm in height. The ring shear specimens had an outer diameter of 100.0 mm and an inner diameter of 70.0 mm. The specimens were consolidated under vertical applied stresses of 10, 20 and 40 kN/m^2. The specimens were then slowly sheared at a rate of 0.00975 mm/min in the shearbox apparatus and 0.096 deg/min in the ring shear apparatus. A Mohr-Coulomb analysis (Figure 7) indicated that the ring shear and shearbox apparatus gave similar effective stress shear strength parameters of c' = 0 and ϕ' = 38^o. These values are similar to those reported by Farrell and Hebib (1998) carried out on peat material from Raheenmore bog.

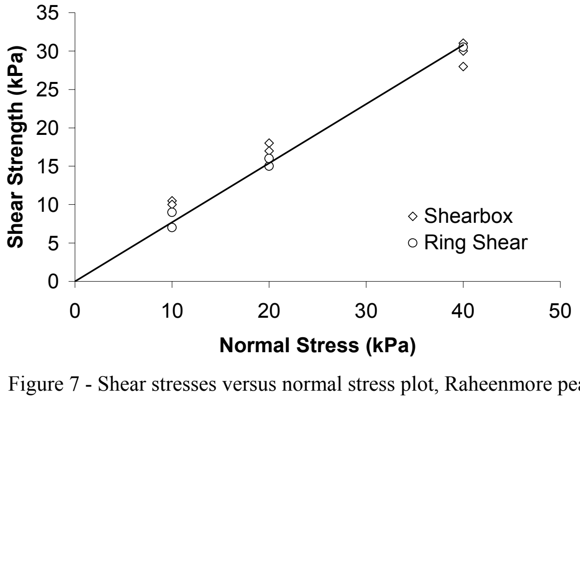

Figure 7 - Shear stresses versus normal stress plot, Raheenmore peat.

The FOS was calculated as F_{max}/P_{hor}, with failure occurring for FOS < 1.0 (Table 3). The FOS for the case of the water depth equal to the overflow level and with the crest of the dam in an unsaturated state was below unity indicating that failure would have occurred if that particular set of circumstances had arisen. An intense rainfall event occurred prior to the failure in December 1998 that would have increased the hydrostatic force acting behind the dam. The possibility of increasing uplift pressure from preferential pathways in the peat could have reduced the FOS even further (Gill, 2005).

Water level	Peat at crest	FOS
At overflow pipe inlet level	Saturated	3.48
At overflow pipe inlet level	Unsaturated following dry weather	0.95
1 m below overflow pipe inlet	Saturated	9.67
1 m below overflow pipe inlet	Unsaturated following dry weather	2.64

Table 3 - Safety factors for different scenarios, Raheenmore.

5 NUMERICAL ANALYSIS

The Edenderry canal embankment was modelled in the finite element computer package PLAXIS 2D (Figure 8).

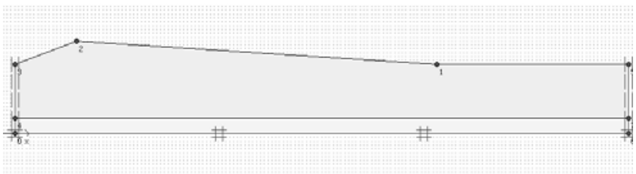

Figure 8 - Edenderry model in PLAXIS.

The boundary conditions were set at a distance of 25 m beyond the toe of the embankment to facilitate the movement of the failed block of peat in the analysis. Standard fixities were assigned; full fixity at the base of the model and roller conditions at the vertical sides. The input parameters are listed in Table 4.

Soil Type		Peat	Glacial Till
Soil Model		Mohr-Coulomb	Mohr-Coulomb
Saturated unit weight (kN/m^3)	γ_{sat}	10	18
Dry unit weight (kN/m^3)	γ_{unsat}	5	15
Horizontal permeability (m/s)	k_x	10^{-6}	10^{-5}
Vertical permeability (m/s)	k_y	10^{-7}	10^{-5}
Young's modulus (kN/m^2)	E_{ref}	400	2000
Poisson's ratio	ν	0.1	0.33
Effective cohesion	c'	0	2
Friction angle (deg)	ϕ'	38	24

Table 4 - Input parameters for Edenderry PLAXIS model.

The back-analysed value of ϕ' from the van Baars calculations was used here. Once the geometry had been established and the material properties had been assigned to each soil cluster, the geometry was meshed to create finite elements with the global coarseness set to medium (Figure 9).

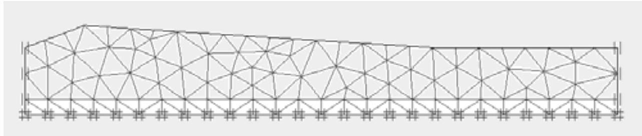

Figure 9 - Finite element mesh of Edenderry canal embankment.

The model contains a non-horizontal soil surface and so gravity loading was used in the calculation phase (by carrying out a plastic calculation where the soil weight multiplier was increased from 0.0 to 1.0) to generate the initial stress field. The water pressures were activated at the same time as the soil weight using the default unit weight of water of 10 kN/m^3. The phreatic water level, for the calculation of the initial pore water pressures, was located along the bog surface under the embankment. Two stages were executed in the calculation phase, namely:

- Gravity loading of the soil was applied as described above.
- The depth of water behind the embankment was increased to near the crest of the dam.

Failure of the soil mass occurred in the PLAXIS computer model following the above steps. Figure 10 shows the predicted ground movements following the Phi-c reduction analysis (the strength parameters entered at the input stage are successively reduced until failure occurs giving FOS = available strength / strength at failure). Figure 10 shows the ground movements predicted by PLAXIS. The slip plane reported by Pigott et al. (1992) is superimposed in Figure 10 for comparison.

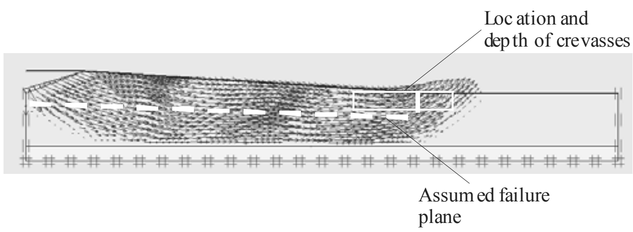

Figure 10 - Predicted ground displacements, Edenderry embankment.

The predicted ground movements involve predominantly lateral displacements of the body of the embankment with most of the movement occurring above the failure plane reported by Pigott et al. (1992). Upward movement of the soil mass occurs to the right hand side of the embankment. Figure 11 shows the effective stresses in the soil mass in the output window of PLAXIS following the calculations phase.

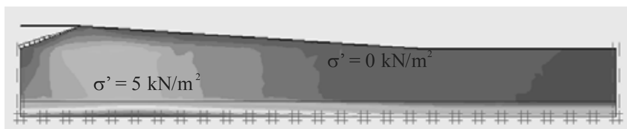

Figure 11 - Effective stresses, Edenderry canal embankment.

The dam at Raheenmore was modelled as shown in Figure 12. Standard fixities were assigned to the boundaries of the model. The input parameters are summarised in Table 5.

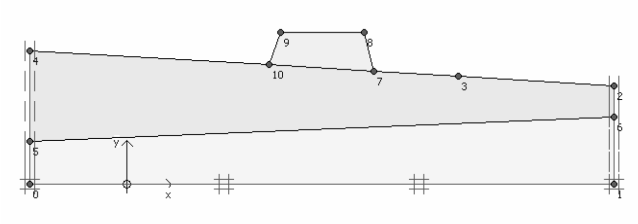

Figure 12 - Raheenmore PLAXIS model.

Soil Type		***In-situ* peat**	**Dam peat**	**Clay**
Soil Model		Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb
Unit weight (kN/m^3)	γ_{sat}	11.4	10.5	17
Horizontal permeability (m/s)	k_x	10^{-6}	10^{-6}	10^{-8}
Vertical permeability (m/s)	k_y	10^{-7}	10^{-7}	10^{-8}
Young's modulus (kN/m^2)	E_{ref}	420	420	2000
Poisson's ratio	ν	0.1	0.1	0.35
Effective cohesion	c'	0	0	0
Friction angle (deg)	ϕ'	38	38	30

Table 5 - Input parameters for Raheenmore model.

The geometry was meshed and the calculation phases carried out in a similar manner as for the Edenderry embankment analysis. Figures 13 and 14 show the directions of the ground movements and the effective stresses in the soil mass, respectively, in the output window of PLAXIS.

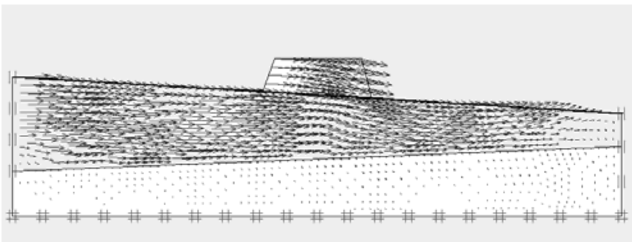

Figure 13 - Predicted ground displacements, Raheenmore dam.

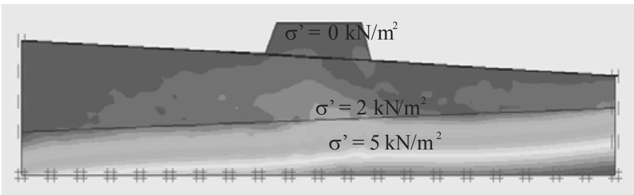

Figure 14 - Effective stresses, Raheenmore dam.

6 DISCUSSION

The van Baars method (2005) was adopted in analysing the Edenderry canal embankment and the Raheenmore conservation dam. Both structures had FOS values of less than unity for the case where the crest material may have been partially dried and the depth of the retained water had risen to the highest level possible. Geotechnical failure of peat embankments or dams generally involves the lateral displacements of large intact peat blocks as opposed to a traditional slip circle. The finite element analysis method has the advantage that it does not require the assumption of the shape or the location of a slip surface. Numerical analyses were carried out for the case study sites using PLAXIS 2D.

The predicted ground movements for Edenderry and Raheenmore concurred with the actual lateral displacements that resulted in the slip failures . Low effective stresses were calculated as expected in the finite element output plots.

7 SUMMARY AND CONCLUSIONS

In general, geotechnical failures of peat embankments and dams occur by the lateral displacement of an intact block of peat material due to reductions in the embankment self weight following partial drying of the crest material and an increase in the active hydrostatic pressures following an increase in the depth of retained water after an intense rainfall event. These factors must be taken into account when carrying out horizontal and vertical stability calculations for peat embankments and dams. Finite element analysis gives a good indication of the likely failure mechanisms and also gives useful information on the low values of effective stress in peat embankments and dams as a result of the high groundwater levels and the low buoyant weight of the peat material itself.

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