Peat unit weight effects on embankment stabilization

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ABSTRACT
Canadian engineers have accumulated extensive experience in stabilizing floating embankments over peat deposits. Nevertheless periodic unexplained failures occur that require remediation. It is the stabilization of these that is the topic of this paper. Peat is a unique soil in that its natural unit weight is only slightly above that of water, so that its submerged unit weight approaches nil. The unit weight increases as peat consolidates under an embankment and therefore will vary in the field from a minimum beyond the toe to a maximum under the centerline. Unit weights also vary naturally and the effect for example of assuming 10.80 versus 11.80 kN/m^3 results in a 100% difference in effective stress and therefore resisting force of the peat. The unit weight parameter selection therefore significantly affects the results of the analyses for remedial measures. The above is illustrated for a low embankment over soft peat. Unit weight variations associated with both Mother Nature and consolidation are calculated, and the effects on the analyses of various failure modes assessed. The paper concludes that field measurements, consolidation effects and sensitivity analyses for peat unit weight are critical to both back analyses of failures and design of stabilizing measures.

RÉSUMÉ
Les ingénieurs canadiens ont accumulé une expérience considérable dans la stabilisation de digues flottant sur des dépôts de tourbe. Pourtant, des défaillances occasionnelles inexpliquées se produisent et exigent une remédiation. C'est la stabilisation de celles-ci qui est le sujet de cet article. La tourbe est un sol unique dont le poids spécifique n'est que légèrement au-dessus de celui de l'eau de sorte que son poids spécifique, lorsque submergée, approche de zéro. Le poids spécifique augmente alors que la tourbe se consolide sous une digue, et celui-ci variera d'un minimum sous le pied à un maximum sous la ligne centrale de la digue. Les poids spécifiques varient aussi de façon naturelle et l'effet, par exemple, de supposer 10.80 plutôt que 11.80 kN/m3 résultera en une différence de 100% de la tension réelle et par conséquent la force de résistance de la tourbe. Le choix du paramètre de poids spécifique affecte de façon déterminante les résultats de l'analyse pour les mesures de remédiation. Ceci est illustré par une tourbe souple pour une digue basse. Les variations du poids spécifiques associées à Mère Nature ainsi qu'à la consolidation sont calculées, et les effets de l'analyse sur les différents modes de défaillance sont évalués. Cet article conclue que les mesures sur le terrain, les effets de consolidation et l'analyse de sensibilité du poids spécifique de la tourbe sont essentiels à l'analyse rétroactive de défaillances et à la conception de mesures stabilisatrices.

1 INTRODUCTION

Embankments have been constructed over peat for centuries, for example sea dykes in the Netherlands (e.g. Buisman, 1936), roads in Ireland (e.g. Munro, 2004) and railway embankments across Canada (e.g. MacFarlane, 1969). Originally based primarily on experience, the design and construction over the past 50 years has developed with various rational and practical approaches. These are basically focused on design for stability against shear failure through the peat, and deformation to address large short term and long term consolidation settlements. The most common applications of embankments floating over peat are highways, railways (Figure 1) and dams (e.g. Hardy, 1968).

Whereas the main challenge in design and construction of fills over peat is usually the original construction, the need to stabilize such an embankment after many years of satisfactory performance is becoming more and more common. This is particularly so for infrastructure that is now required to perform under new loads and conditions. Examples are trains that have become considerably heavier, longer, more frequent and faster than in the past. Another example is the need to expand infrastructure, for example installation of buried services near the shoulder or toe of embankments, or widening of highway embankments. It is no longer enough to simply understand the basic principles for new construction over peat, such as undrained parameter selection, deformation prediction, stage construction sequencing or geosynthetic reinforcement.

Figure 1. Railway embankment stabilization over peat

When it comes to remedial work for existing embankments over peat, this is usually in response to an unexpected increase in the rate of movement in some part of the embankment surface. Assuming that this occurs well after the original construction, then primary consolidation will be complete (excess pore pressures dissipated) and secondary consolidation will be underway, albeit at an ever decreasing rate. Therefore any new increase in rate of movement that the embankment experiences must be a result of shear movement, whether as a slow creep or sudden displacement.

2 ANALYTICAL METHODS

To characterize the peat behaviour with respect to strength, two approaches are available: total stress using undrained parameters versus effective stress using drained parameters. The former is typically applied to the case during construction when pore pressures are in excess of hydrostatic. The use of undrained parameters simplifies the analysis since pore pressures do not need to be measured or estimated. An undrained approach is typically applied to an existing embankment only if the new deformations are associated with excess pore pressures. A total stress analysis would also apply to any new loads from stabilizing measures, for example stability of the flanking berm itself. Effective stress analyses, on the other hand, where the strength varies with stresses on the shear plane, are typically applied when the excess pore pressures are measured or predicted with confidence. In many cases when new movements within an existing embankment appear to occur without excess pore pressures, then drained parameters are applied.

Once the existing conditions have been defined, a back analysis of the failure is typically carried out to confirm reasonable average soil strength parameters, using assumed reasonable failure modes and groundwater conditions as indicated by observations or monitoring. The parameters are then applied to the remedial works. These analyses are typically carried out using limit equilibrium analysis methods.

The state of the practice in embankment design over soft ground to a large extent still depends on limit equilibrium methods to estimate a safety factor. This method allows assessment of many essential sets of variables, including stratigraphy, geometry and slip failure. However, it does not address either peat property changes during consolidation or deformation behaviour. Remedial designs for flanking berms are easily developed. Unlike for inorganic soils, the results are very sensitive to the selection of unit weight of the peat.

The embankment can also be analyzed using a stress-deformation method applying finite element code. This method addresses both time dependent deformation and stresses simultaneously, and therefore requires both deformation and strength parameters. The modeling process considers changes in soil parameters due to deformation of soils caused by change in stresses. Soil mobilization, principal stress rotation and hence modes of failure are also taken into consideration. Furthermore, changes in soil hydraulic conductivity during consolidation as well as non-linear stress strain behaviour, large strain effects and submergence effects are taken into account in the process (Bo & Choa 2004; Bo 2008). In addition, a different soil model can be assigned to each type of soil. Therefore more realistic deformation behaviour can be modeled, including the various stages during construction. Safety factors are calculated using the strength reduction method through phi-c reduction.

3 EMBANKMENT PROBLEMS

Because peat goes through dramatic changes as it consolidates, design of stabilizing measures for an existing embankment needs to deal with a material that has developed a wide range of properties, from its original weak and highly compressible nature at a point far from the fill, to a maximum strength and minimum compressibility under the embankment centerline. In addition, there is the natural variability inherent in peat deposits to deal with.

There are many potential causes and failure modes which may be experienced by existing embankments, for example as a result of the following changes:

- loads from traffic that is heavier, faster and more frequent than before.
- widening or raising embankments.
- disturbance of the fill materials and/or the underlying peat, for example as a result of installing buried utilities.
- loss of toe support as a result of ditching or erosion.
- increase in effective stress under the fill as a result of a lowered water table, for example as a result of drainage measures or general climate change
- additional seepage pressures as a result of uphill water infiltration (e.g. Fabius et al, 1999).
- changes in dynamic loading. These effects can be exacerbated under certain stratigraphic geometries due to resonance (e.g. Katzenbach and Ittershagen, 2004) or other magnification processes (e.g. Hendry et al, 2007).
- decomposition of the peat (e.g. Landva et al, 1983).
- weakening of structures within or below the fill as a result of deformation or aging, for example culverts, reinforcement, or piling.

These changes can result in a wide variety of effects, from localized depressions to large scale movements. A variety of failure mechanisms may be involved.

4 STABILIZATION TECHNIQUES

Construction of fills floated over organic deposits requires special techniques that differ from those supported on inorganic soil. Due to its low undrained shear strength and low effective stress resulting from low initial unit weight, the load carrying capacity of peat can be extremely low under both undrained and drained conditions. In addition to the excessive vertical displacement occurring due to its initially low effective stress and high primary and secondary compression indices, there is also considerable lateral displacement due to lateral stresses on the peat. The latter typically develops near the slope toe, particularly due to principal stress rotation.

Construction methodologies have been developed to minimize settlements and maximize stability. These include:

- preservation of the root mat
- a slow rate of fill application
- construction during frozen conditions
- sequencing fill placement to allow consolidation between lifts
- advancing flanking berms ahead of the central fill
- applying geosynthetics for load spreading.

In addition, the construction can be optimized with respect to schedule and risk reduction. This is accomplished by adjusting the method based on results of monitoring changing conditions during construction, for example deformations, pore pressure and undrained shear strength.

Many stabilization designs have been successfully applied in the industry. These can be generally categorized as making weight adjustments, adding reinforcement or improving the foundation materials. For floating fills that have been stable for a period of time but require stabilization due to changed conditions, the most practical and economical solution is often through slope flattening or the construction of a low flanking berm. Whereas the berm provides additional toe resistance it also induces new settlement of the shoulder and slope of the embankment due to stress influence and overlapping. This solution requires adequate space as well as fill availability.

In some cases due to constraints such as space, access, schedule, deformation tolerances or environmental conditions, a flanking berm may not be suitable, and one of several alternate designs may be considered. In other cases where pore pressure is an issue, for example due to changes in drainage or infiltration, drainage facilities may also be a solution. This paper, however, focuses on the most commonly used solution, namely increasing the stability with flanking berms.

5 PEAT CHARACTERISTICS

For design of remedial measures which address instability of a fill over peat, key input data required for the peat are its physical parameters including unit weight, and strength. In addition, compressibility data is key to stress-deformation analysis methods.

5.1 Physical Characteristics

The term ‘peat’ is often used to include organic soils that may range from jelly-like organic silts and very soft organic clay “mud” to extremely coarse-fibrous meshes of woody remains and fibres. Characterization of peat has been somewhat difficult due to a few reasons. Peat can behave very differently depending upon its state of decomposition. A highly decomposed peat and significant clay content behaves like clay whereas peat with extremely coarse-fibrous meshes with low ash content may behave like granular soil. Conventional in-situ tests like the field vane test (FVT) and the cone penetrometer test (CPT) do not measure well on fibrous peat (Rahadian et.al 2001). Sample collection is also somewhat difficult due to disturbance occurring during sampling and the inability to maintain the water content during the extraction of the sample. Nevertheless, engineers have managed to extract reasonable quality samples with the help of the Hiller borer, the Davis sampler and piston samplers.

There are many classification systems with different definitions of what constitutes “peat” (Landva et.al, 1983; Hartlen & Wolski, 1996). Unfortunately, these systems are not very consistent (Leroueil & Rowe 2001). Classification of peat is generally carried out to define the degree of humification applying the Von Post classification test. Peat can also be classified in accordance with ASTM D4427. Both water content and ash content are usually measured to characterize the type of peat. Water content is measured after drying out the peat under low temperature in the laboratory oven. Peat usually has a low specific gravity due to its lightweight of organic matters. Loss on ignition is also measured to characterize the peat.

Peat typically has a very high natural water content (100-2000%), high void ratio (usually 5-15, but may be up to 25) (Hanrahan, 1954) and very high compressibility (Leroueil & Rowe 2001). Specific gravity of peat ranges from 1.5 to 1.8 with bulk unit weight ranging from 9 to 12 kN/m^3 (Rahadian et.al 2001). The unit weight correlates closely with ash, moisture and gas contents (IRE 2000; Skempton and Petley 1970).

Typical characteristics of peat are summarized in Tables 1 and 2.

5.2 Strength Characteristics

Due to a high void ratio and compressibility as well as fibre content, it is usually not practical to obtain realistic strength parameters from conventional triaxial testing, which yield high peak angles of drained shear strength ranging from 45 to 55 degrees (Adams 1961; Edil & Dhowian 1981; Rowe et.al 1984a; Leroueil & Rowe 2001). Nevertheless, Landva (1980) has had success in obtaining reliable strength parameters using a ring shear apparatus whereas Rowe et al. (1984b) and Rowe & Rowe and Mylleville (1996) have obtained clearly defined failure envelopes using the Norwegian simple shear apparatus (Leroueil & Rowe 2001). A triaxial test carried out on Nerengbengkel peat in Indonesia shows no peak strength until 30 % strain. Rahadian et.al (2001) reported peak angles of shear strength of 17 to 39 degrees with 0-4 kPa effective cohesion from a triaxial test and peak angles as high as 43 degrees from a direct shear test on Indonesia peat. Landva & LaRochelle (1983) carried out large strain ring shear testing and concluded that at large strains which remove fibre reinforcing effects, the peak angles of drained shear strength for peat is typically in the range of 30^{o} to 32^{o}.

Table 1. Typical Physical Characteristics of Peats (*MacFarlane 1969, **Riley and Michaud, 1989)

Description	Water Content (%)	Ash Content (%)	Natural Unit Weight (kN/m^3)
Black Fibrous Peat*	300-650	15-40	10.2 – 10.5
Fla. Peat*	485 - 910	17	9.3
Ireland*	340 - 1465		9.3 – 10.1
Vancouver*	500 - 1500		8.6 – 11.8
Fine Fibrous*	145 - 480		9.0 – 12.2
Ishikari Peat*	155 - 810	17 - 43	9.3 – 11.0
Churchill Area Peat*	205		11.0
St. Elie d'Orford, Que.*	200 – 890	7 - 13	9.2 – 10.1
Napierville, Que.*	300 - 650	15 - 40	10.2 – 10.7
Medium Peat*	240 - 340		9.4 – 11.1
Von Post H1-H3**	270 – 1900 (mean = 733)	1.5 - 19.7 (mean = 7.6)	9 – 13 (mean = 10.2)
Von Post H4 +**	270 – 2400 (mean = 809)	1.2 - 19.7 (mean = 6.8)	9.4 – 11 (mean = 10.2)

With respect to undrained strength, Rahadian et.al 2001 carried out vane shear tests on Indonesia peat using two types of field vane equipment (Farnell and NGI) with two different sizes of vane diameter. They noted strain-hardening behavior with no peak strength until very large strains. A larger vane produced a wider range of scatter values (Rahadian et.al 2001), which contradict to the findings for Canadian peats reported by MacFarlane (1969). Rahadian et.al (2001) reported that the strength of peat does not increase with depth based on their field and laboratory measurement of undrained shear strength tests on Indonesian peat. Based on the CPT carried out side by side with FVT using Farnell and NGI equipment, Rahadian et.al reported cone factor N_{kt} values of 13.2 and 18.4 for NGI and Farnell equipment respectively for Indonesian peat. In general, the undrained shear strength of peat varies inversely with its water content and directly with its ash content (Wyld 1956).

Table 2. Typical Drained Strength Characteristics of Peats (Leroueil and Rowe, 2001)

Description	Water Content (%)	c' (kPa)	φ' (°)
Fine Fibrous Peat	527	1.0	26
Fine Fibrous Peat	656	1.2	26
Course Fibrous Peat	680	2.5	28
Course Fibrous Peat	1450	0.5	29
Sphagnum Peat	1200-1500	2.5	27
Sphagnum Peat	1200-1500	3.5	33

Selection of suitable strength parameters for either undrained or drained analyses can be difficult, primarily because the fibres typically found in peat affect any testing procedure. The fibres can have a significant reinforcing effect during the initial straining of an embankment, however their effect reduces considerably with large strains as the fibres gradually realign parallel to the shear plane. The latter is often the case when embankment movement has occurred, and remedial measures are designed. Other parameters which are difficult to select yet have a significant effect on the results, unlike for inorganic soils, are void ratio and unit weight.

5.3 Unit Weight

When analyzing failures of fills over peat, the unit weight of the peat itself is an important factor. This is particularly so because for most cases, natural peat is saturated and has a very low effective stress. Furthermore, peat's natural gas content may result in a unit weight less than that of water. Low ash content peats typically have saturated unit weights ranging from less than 10 to 11 kN/m^3, providing submerged unit weights of less than 0.2 to 1.2 kN/m^3. Natural variations of 0.5 kN/m^3 over short distances are not uncommon. After consolidation under fills, the peat unit weight typically increases by up to 2 kN/m^3, depending on the degree of consolidation and applied stress. The unit weight variation with moisture content, a common classification measurement, is illustrated on Figure 2, for specific gravities of 1.50 and 2.75 for organic and inorganic matter, respectively. For many Canadian peats, natural moisture contents are in the 500% to 1000% range (Table 1).

These variations can be difficult to detect with field or laboratory testing. The assumption for the initial natural bulk unit weight, for example 10 versus 11 kN/m^3, is a 600% variation in the submerged unit weight. . When back-analyzing the failure of an existing fill, and designing remedial measures, the unit weight assumption can result in an even bigger variation due to the compressed peat under and near the embankment. And yet this variation is rarely adjusted for in practice.

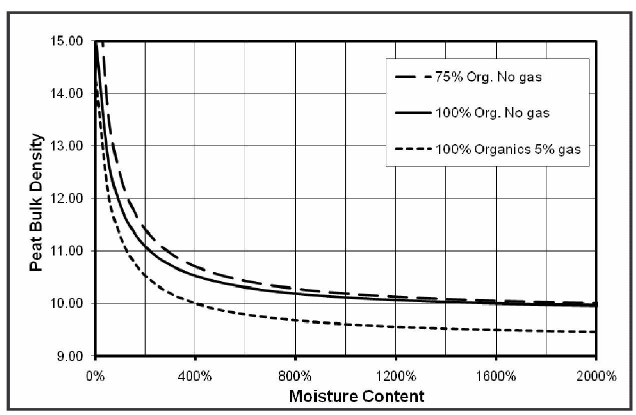

Figure 2. Peat Unit Weight Versus Moisture Content

The unit weight impacts the analysis in 2 ways. Firstly, it provides a resisting force against circular or lateral sliding through the peat. Secondly, when analyzing using drained parameters (assuming a cohesion c' of nil), the strength is directly proportional to the effective stress for a specific failure plane. A 600 % range in unit weight is therefore equal to a 600 % variation in strength for the shear zone located outside the embankment.

6 ANALYSIS OF STABILSTABILIIZING MEASURES

It is common practice to analyze embankment stability over weak ground to achieve a reasonable factor of safety by applying limit equilibrium methods under plane strain conditions. With a two dimensional subsurface model, material properties can be varied both vertically and horizontally. However, most engineers apply constant parameters along the horizontal plane due to limited spatial data available and the difficulty in identifying the boundary locations for variations. For most natural young soils deposited under the same geological history and environment, horizontal variations are rare. However for organic deposition, natural horizontal variations are more common, since these vary with effects of localized variations in the original vegetation type and growth rate, as well as small variations in contamination by localized inorganic soil deposition. Furthermore, given their very low submerged unit weight, their properties are changed dramatically by small fluctuations in the water table as well as snowfall, both of which can vary over short distances.

For an initial analysis of a new embankment over peat, normally carried out for total analysis, the peat unit weight is usually assumed to be a constant, and its actual value has little effect on calculation results. However for back analysis of existing embankments and the design of flanking berms, both under drained conditions, this peat unit weight value has a significant impact on results. Nevertheless, such variations are rarely considered for routine analyses of stabilizing measures.

The analyses outlined in the following sections demonstrate the effect of unit weight variation on the results of slope stability analyses and the remedial design.

6.1 Methods of Analysis and Software Used

Both limit equilibrium and phi-c reduction methods were applied in this study. Slope stability was analysed under plane strain conditions in the 2-dimensional domain. For the limit equilibrium method, both force and moment equilibrium was analyzed applying several methods such as Bishop (1955), Janbu (1973) and Morgenstern and Price (1965). The minimum safety factors are reported here. Slope/W, version 6.22 developed by Geo-Studio, was used. For the phi-c reduction method, a stress deformation model was run to failure by reducing the phi and c values from the input values, and safety factors were determined from the ratio of available strength to required strength. PLAXIS Version 8.2 finite element software developed by Plaxis BV was used for finite element modeling (Figure 3). A soft soil creep model was applied to the peat, which incorporates secondary consolidation effects.

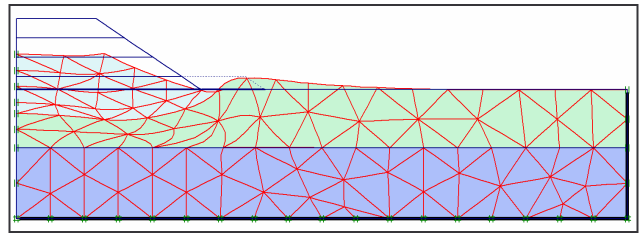

Figure 3. PLAXIS Output – Deformed Mesh

6.2 Input and Analysis Options

The base case for an existing embankment floating over peat was established using finite element modeling and the input data summarized in Table 3. A 5 m thick deposit of saturated peat (moisture content 600%) was allowed to consolidate under self weight for a substantial period of time (to model the effects of the ongoing long term secondary consolidation), and subsequently under the weight of a 5 m thick cohesionless fill placed in 3 stages. After primary consolidation, the top of the deformed embankment was levelled to create the base case of a 1.9 m high existing embankment with 1.5 horizontal to 1 vertical side slopes.

7 RESULTS AND DISCUSSIONS

The net result was a model with a subsurface stratigraphy commonly found in Canada under existing railway and highway embankments. Peat strains are up to approximately 80%. The output data also provided the distribution of unit weight variations under the embankment (Figure 4).

Additional analyses illustrated that much smaller unit weight variations occur when the peat strain is less than 50%.

The embankment model was then back analysed using limit equilibrium methods for a safety factor of 1 to represent an embankment experiencing excessive shear movements. Excess pore pressures were assumed to be nil. A 30^{o} angle of peak shear strength (Φ') was assumed for the cohesionless fill as well as for the peat, with an effective cohesion of nil. These parameters are commonly applied to loose granular fills, as well as peat at large strains where the effect of fibres is removed (Landva and LaRochelle, 1983). Possible variations were then considered for the unit weight. It was found that for a constant unit weight assumption, the unit weight of the peat at failure must be 9.85 kN/m^3. Other unit weights are possible if the peat or fill strength parameters are varied.

Table 3. Plaxis Input Data

		Fill	Peat	Underlying Soil
Soil Model		Mohr-Coulomb	Soft Soil Creep	Mohr-Coulomb
Unit Weight γ (kN/m^3)	Above Water	18.5	N/A	17.0
	Below Water	20.0	10.5	21.0
Apparent Cohesion *C'* (kN/m^2)		N/A	N/A	N/A
Initial Void Ratio e_o		N/A	10	N/A
Peat angle of shearing strength *Φ'* (°)		30	30	33
Young's Modulus *E* (kN/m^2)		8000	N/A	1.2×10^5
Poisson Ratio □		0.3	N/A	0.3
Hydraulic Conductivity *k* (m/s)	Vertical	1×10^{-5}	1×10^{-4}	5×10^{-4}
	Horizontal	1×10^{-4}	N/A	5×10^{-4}
Hydraulic Conductivity Reduction Ratio		N/A	2	N/A
Compression Index		N/A	6.5	N/A
Re-Compression Index		N/A	0.65	N/A
Secondary Compression Index		N/A	0.26	N/A

In order to evaluate the sensitivity of unit weight variation on stabilizing berms, the following possible models were analysed for a low 0.6 m thick flanking berm. Such a berm is often used as a practical and economical stabilization option. The analysis was carried out for a constant 9.85 kN/m^3 unit weight found from the back analyses, as well as for an alternative assumptions where the unit weight is varied under and near the embankment based on the distribution found from the stress-deformation analyses. The results are tabulated in Table 4 with flanking berms for safety factors of 1.3 and 1.5. The unit weight has a significant impact on the berm design, with a lower unit weight (and higher peat strength) providing a smaller berm (Figure 5). For the same initial unit weight assumption, the constant versus variable unit weight assumption has only a mall impact on berm design.

Table 4. Flanking Berm Design Results

Peat Unit Weight Assumption (kN/m^3)	Peat Phi for FOS = 1.0	Berm Width for FOS = 1.3	Berm Width for Safety FOS = 1.5
Constant 9.8	38.0	8.6	11.0
Constant 9.95	30.0	10.6	13.4
Constant 9.90	33	9.8	12.3
Variable (initial 9.9)	30.0	10.2	13.0
Constant 10.0	28.5	10.8	13.8
Constant 10.5	20.5	13.4	16.2
Variable (initial 10.5)	20.0	13.7	16.3
Constant 11.0	16.5	13.6	17.2
Constant 11.5	14.0	14.0	17.7
Constant 12.0	12.5	14.4	17.7

The above results have been based on limit equilibrium analyses. The same embankment was analyzed using finite element modeling (PLAXIS), with and without stabilizing berms using the parameters noted in Table 3. This method takes full account of several variations such as including soil parameters in both horizontal and vertical directions, change of soil parameters and geometry during consolidation process, mobilized soil strength at various stress levels, principal stress rotation, mode of failure, non-linear stress-strain relationships, submergence effect and non-uniform strain (Bo & Choa 2004). Therefore the method provides more realistic behaviour of soil deformation. Figures 6 and 7 show PLAXIS output results for horizontal and total displacements.

The results indicate peat with a 30˚ peak angle of shearing resistance and an initial unit weight of 10.5 kN/m^3 does not deform excessively. In addition, the majority of the soil elements did not mobilize beyond the yield strain. To verify the model as one representing the

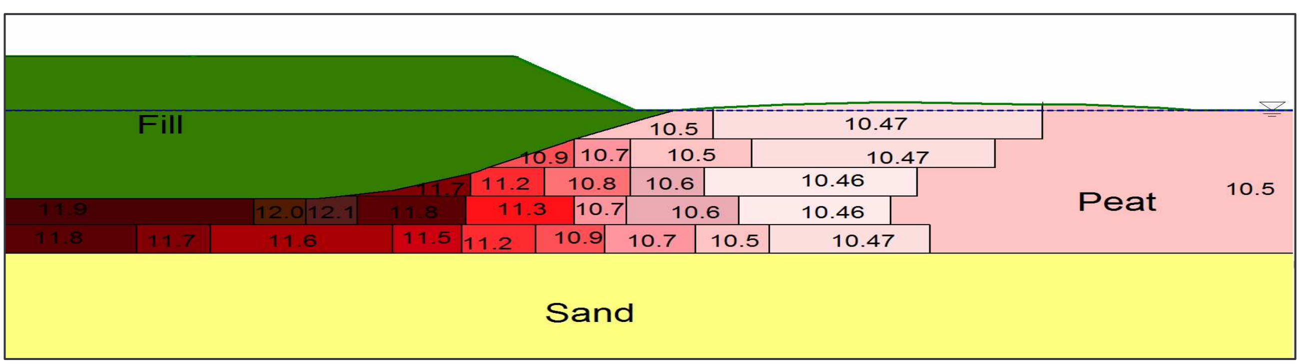

Figure 4. Subsurface Model for Fill Over Peat

observed failure would require adjusting the unit weight and/or strength parameters until deformation matches that observed.

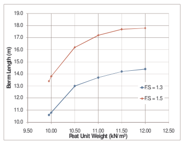

Figure 5. Peat Unit Weight Effect on Berm Design

A significant benefit of using PLAXIS is that it requires input of only a single initial unit weight, thereafter the program updates changes in material parameters. As principal stress rotation, mobilization of strength and mode of deformation are considered much of the uncertainty is eliminated.. Moreover with PLAXIS both soil deformation under elastic, consolidation, seepage conditions as well as stability under static and dynamic conditions are modeled in one go.

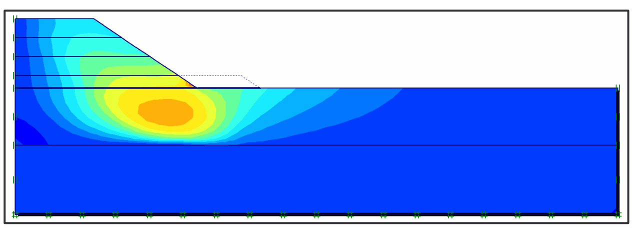

Figure 6. PLAXIS Output – Horizontal Displacement

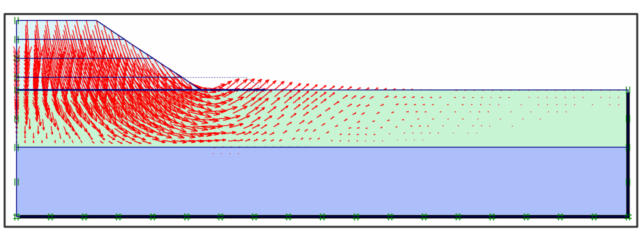

Figure 7. PLAXIS Output – Total Displacement

8 CONCLUSIONS.

The initial design of an embankment is generally insensitive to the peat unit weight since the design is usually based on undrained conditions for the peat. For analyses of an existing embankment after complete pore pressure dissipation, however, the unit weight assumption has a significant effect on the results. Yet an accurate modeling of these are rarely taken into account in practice due to practical difficulties in assessing natural variations as well as variations resulting from varying stresses under the embankment. This paper has studied the sensitivity of the design of a stabilizing flanking berm to the peat unit weight assumption.

The results of the modeling and analyses indicate the following:

- The design of flanking berms for existing unstable embankments using limit equilibrium methods and drained strength parameters selected based on back analyses is highly sensitive to the unit weight assumption.
- To illustrate, for a typical 1.9 m high embankment over a peat deposit with a 600 % moisture content, and where peat strength parameters are based on back analysis, the unit weight assumption can lead to a 50% change in flanking berm width.
- On the other hand, if the peat strength parameters are well known (without selection based on back analysis), then the unit weight selection (whether constant or variable) has little effect on the berm design.
- A finite element method requires input of only a single unit weight of peat as the program updates changes in soil parameters and geometry including, unit weight, during stage construction and the consolidation process.

In conclusion, the design of a stabilizing flanking berm for an embankment over peat applying limit equilibrium analysis requires an accurate characterization of unit weight, in terms of initial natural unit weight, natural variations and variations resulting from peat consolidation. Unit weight assumptions simplifying the conditions can lead to unsafe designs. FEM modelling is the most accurate way to assess unit weight for back analysis of unstable embankments and design of flanking berm.

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