Proceedings 10th NUMGE 2023
10th European Conference on Numerical Methods in Geotechnical Engineering
Zdravkovic L, Kontoe S, Taborda DMG, Tsiampousi A (eds)

https://doi.org/10.53243/NUMGE2023-85

Calibration of constitutive models for finite element analyses of embankments on peat

M. D'Ignazio[1,2] , S. Sainio[1,3] , T. Länsivaara[1], H. K. Engin[4]

[1]*Department of Civil Engineering, Tampere University, Tampere, Finland*
[2]*Ramboll Finland Oy, Tampere, Finland*
[3]*Sweco Finland Oy, Tampere, Finland*
[4]*Norwegian Geotechnical Institute, Oslo, Norway*

ABSTRACT: Part of the Finnish railway network is built on soft organic soil, including peat. Geotechnical design in peat presents challenges when evaluating stability and deformations under increased traffic loads or when a railway or road expansion is planned. The evaluation of shear strength of peat is not straightforward. Moreover, peat generally shows high compressibility and thus deformations shall be also evaluated along with shear mobilization under embankments. This can be done utilizing finite element analyses. In Finland, Field Vane test is generally used to evaluate undrained shear strength, which does not provide information of soil stiffness. This study focuses on the calibration of peat model parameters for some of the most used FE soil models, including Mohr-Coulomb, Hardening Soil and NGI-ADP constitutive models in PLAXIS. Data from a railway site in Northern Finland was used for the calibration. The data consists of undrained triaxial compression and direct simple shear tests. The calibration parameters are meant to be used in preliminary undrained stability and serviceability calculations of embankments on Finnish peat.

Keywords: peat; modelling; finite element; stability; embankments

1 INTRODUCTION

Peat-producing ecosystems are found throughout the world, and peat deposits constitute 5-8% of the land surface of the earth (Mesri and Ajlouni, 2007). Peatlands are also widespread in Finland and Northern Europe. In particular, peatlands in Finland account for 33.5% of the land surface (Forsman et al., 2018).

Peat belongs to the category of geomaterials and according to ASTM (2013) can be classified as a soil material with more than 75% organics, measured on dry mass basis. Such a material is generally fibrous in nature and can reveal its structure in different states according to the degree of decomposition of the organic matter. The most used classification system for peat is the Von Post (1922), which is based on the degree of decomposition (H1-H9). Fibres in peat material are in the forms of decomposed wood remains, leaves, stems and leaf stalks, rootlets, rhizoids and any other elongated plants or plant remains (O'Kelly 2017).

Peats are challenging materials in geotechnical engineering practice, with many aspects of their behaviour seemingly remaining enigmas (O'Kelly 2017). The mechanical behaviour of peat is complex. The presence of fibres, the strong susceptibility for creep, high natural water content, low strength and stiffness and biological and chemical degradation of the material make designing infrastructure on a peat foundation an engineering challenge. Moreover, in soil mechanics, material models and calculation methods focus on sands and clays, while little attention is given to organic soils. In practice, calculation methods for sands and clays are adopted for modelling peat despite the significant differences in geological origin, structure and behaviour.

In general, peats exhibit significantly higher effective strength parameters (i.e. friction angle, normalized undrained shear strength) and higher hydraulic conductivity compared to silts and clays (e.g. Mesri and Ajlouni 2007; Zwanenburg and Jardine 2015; O'Kelly 2017). This is possibly due to the presence of fibres that, in an intact state, have relatively higher tensile strength than the peat matrix and provide conduits for the preferential flow of water through the bulk material. In addition, the fibre deposition and subsequent large vertical strains experienced during the one-dimensional consolidation in situ produce an inherent structural cross-anisotropy. This will constitute a challenge when selecting the most appropriate laboratory test set-up to determine the basic parameters and calculation methods to evaluate the performance and safety of structures and infrastructures.

Part of the Finnish railway network is built on peat areas. Ensuring the safety of such infrastructures often results in costly geotechnical design owing to the modelling limitations of this challenging geomaterial.

Stability analyses of embankments on peat are often based on the undrained shear strength (s_u). Nevertheless, the high compressibility would suggest that, along with total safety factors, deformations would deserve attention. In practice, the modelling of deformations often is carried out employing Finite Element (FE) analyses. The calibration of the constitutive models requires a comprehensive set of laboratory tests (e.g., triaxial, direct simple shear, oedometer, etc..). In Finland, these are often lacking in many projects, since the determination of s_u is based on Field Vane test measurements, which do not provide information on soil moduli.

This study focuses on the calibration of FE model parameters for Finnish peat for three soil constitutive models: Mohr-Coulomb, Hardening Soil and NGI-ADP. Undrained triaxial and direct simple shear (DSS) data from a railway site in Northern Finland are used to establish model input parameters. The results of this study shall serve as a basis for preliminary FE stability and serviceability calculations under short-term loading of railway embankments on peat.

2 SITE DESCRIPTION AND CHARACTERIZATION OF RISTIJÄRVI PEAT

The site object of study is described in detail in Sainio (2022). The site is located in Northern Finland, near the Municipality of Ristijärvi, along the railway track between Kontiomäki and Pesiökylä (Figure 1).

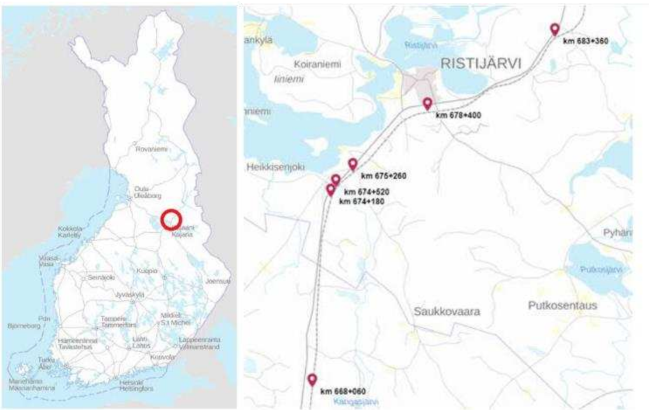

Figure 1. Test site location at Ristijärvi, Finland (after Sainio, 2022).

A comprehensive site investigation campaign has been carried out to characterize the peat layer along the existing railway track. Testing included index tests, piezocone (CPTu), Field Vane (FV), undrained isotropically consolidated triaxial compression (CIUC) and direct simple shear (DSS). Samples were collected from three cross-sections over ≈1 km stretch of the railway track.

The subsoil consisted of 2-4 m-thick peat layer over 1-2 m of soft clay, resting on top of a sand moraine deposit. Sampling and testing were carried out through the embankment, below the embankment counter berm and from the side (virgin peat, free field). Samples were collected using 50 mm, 86 mm and 100 mm diameter piston samplers.

The humification level of the Ristijärvi peat varies significantly, between Von Post H3 and H9. Water content varies between 412% and 1016%, with unit weight of 9-13 kN/m^3. There seems to be a correlation between the Von Post index and the water content, with H3 samples (raw peat) showing the highest water content and H9 samples (decomposed peat) showing the lowest.

The undrained shear strength from FV and CPTu in the virgin peat and under the counter embankment was found to vary between 5 and 35 kPa, with no distinct trend with depth (Figure 2). Field Vane test measurements were reduced by a correction factor μ=0.5 (Liikennevirasto, 2018). A bearing capacity factor N_{kt} = 15 gave s_u in line with the corrected FV measurements.

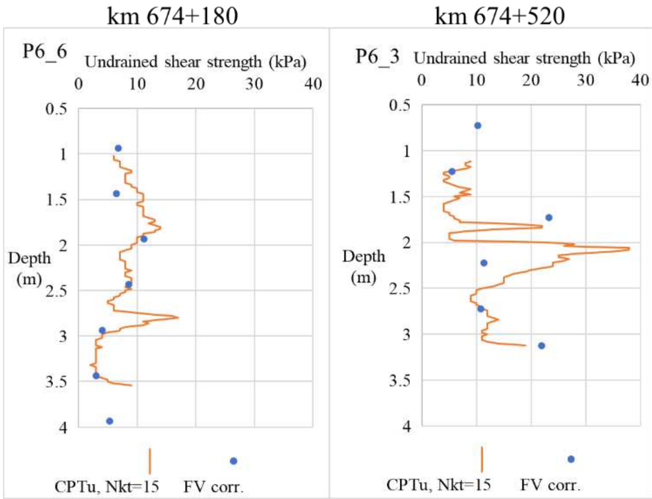

Figure 2. Undrained shear strength of peat from CPTu and reduced FV (μ=0.5) measurements from two locations along the investigated railway track.

The undrained shear strength from DSS tests varies between 2.5-5.2 kPa in the virgin peat, 7 and 15.3 kPa below the counter berm and 22.4 and 24.3 kPa under the embankment. Vertical consolidation stresses applied in the tests vary between 6 and 51 kPa.

Triaxial CIUC tests suggested s_u = 4.2 kPa in the virgin peat, s_u = 4.5-14.1 kPa below the counter berm and s_u = 37.6-38.8 kPa below the embankment. Applied cell pressure in the tests varies between 7 and 66 kPa.

The s_u values from CIUC and DSS are in line with FV and CPTu measurements.

The stress-strain behaviour generally shows strain-hardening until the maximum stress. For some samples, strain-softening was observed (Figures 3 and 4).

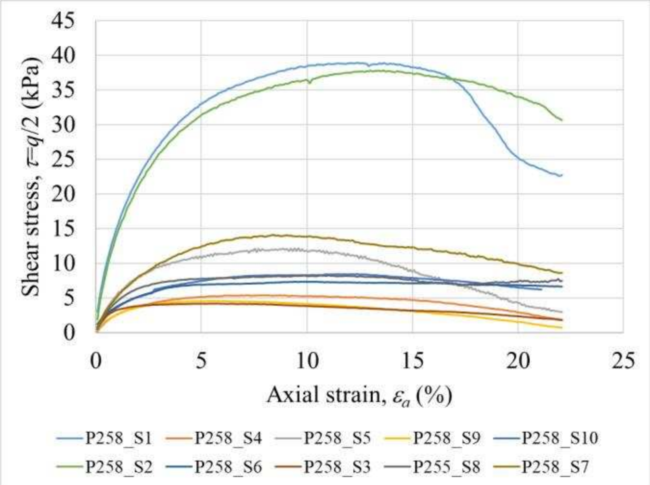

Figure 3. Stress-strain relationships from CIUC tests on Ristijärvi peat.

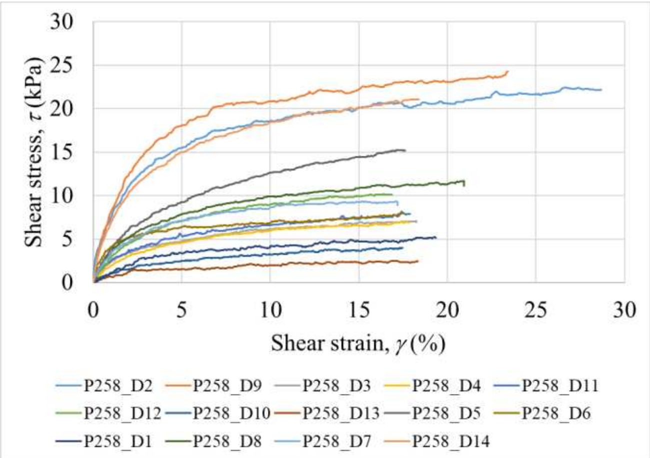

Figure 4. Stress-strain relationships from DSS tests on Ristijärvi peat.

The measured s_u appears to decrease with increasing water content, in line with the framework of critical state soil mechanics, as also observed by e.g., Amaryan et al. (1973) and Long and Boylan (2012) and as suggested by the Finnish embankment stability guidelines (Liikennevirasto, 2018) (Figure 5).

The normalized s_{uDSS}/σ'_v varies in the range 0.36-0.77, with a mean value of 0.53. If s_{uDSS} is defined at 5% shear strain, the values are in the range 0.22-0.47, with a mean value of 0.36. In engineering practice, s_{uDSS}/σ'_v = 0.4 is assumed to estimate s_u under the existing embankment, in the absence of test data (Liikennevirasto, 2018). For soft inorganic Finnish clays, $s_{uDSS}/\sigma'_v \approx 0.24$ (D'Ignazio et al., 2016; D'Ignazio et al., 2021). For organic normally consolidated sulphide Finnish clay from Murro, Western Finland, $s_{uDSS}/\sigma'_v \approx$ 0.33-0.36 (D'Ignazio, 2016; D'Ignazio and Länsivaara, 2016).

The normalized s_{uCIUC}/σ'_v varies in the range 0.35-0.84, with a mean value of 0.58. If s_{uCIUC} is defined at 5% shear strain, the values are in the range 0.32-0.63, with mean value of 0.47. Therefore, the ratio s_{uDSS}/s_{uCIUC} is in the range 0.91-1.03 based on the maximum shear stress criterion and 0.69-0.76 based on the 5% shear strain criterion. The latter values are in line with typical values for clay (e.g. D'Ignazio, 2016).

Sainio (2022) observed how for Ristijärvi peat, the normalized s_u values do not show any clear correlation with index properties or Von Post index.

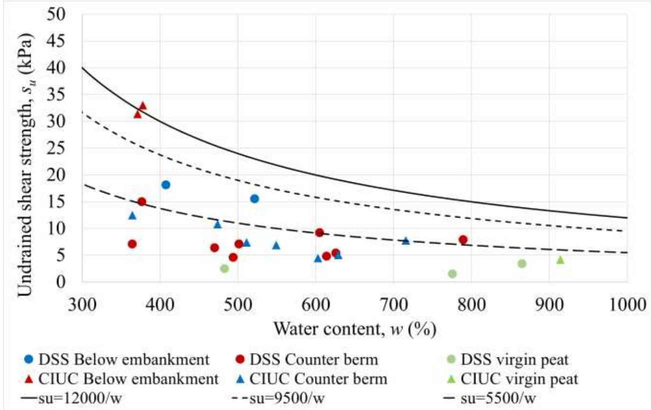

Figure 5. Undrained shear strength (maximum shear stress) from CIUC and DSS tests on Ristijärvi peat versus water content and s_u=f(w) according to Liikennevirasto (2018).

3 SOIL CONSTITUTIVE MODELS

3.1 Mohr-Coulomb model

The Mohr-Coulomb (MC) model is a simple linear elastic perfectly plastic model. The linear elastic part of the MC model is based on Hooke's law of isotropic elasticity. The perfectly plastic part follows the Mohr-Coulomb failure criterion.

The yield surface is fully defined by model parameters and not affected by plastic straining. For stress states within the yield surface, the behaviour is purely elastic.

The MC model requires parameters that are generally familiar to most geotechnical engineers. These are the Young's modulus E' (drained) or E_u (undrained), alternatively the shear modulus (G), the Poisson's ratio ν' (drained) or ν_u (undrained), the undrained shear strength s_u or the drained effective cohesion (c'), friction angle (ϕ') and dilatancy angle (ψ).

3.2 Hardening Soil model

The Hardening Soil (HS) model is an advanced model for simulating the behaviour of both soft soils and stiff soils (Schanz, 1998). When subjected to primary deviatoric loading, soil shows a decreasing stiffness and simultaneous irreversible plastic strains develop. In the special case of a drained triaxial test, the observed relationship between the axial strain and the deviatoric stress can be well approximated by a hyperbola. The hyperbola tends asymptotically to an upper limiting deviator stress at failure. Hence, the failure criterion is defined so that the maximum deviator stress is lower than the asymptotic value to obtain a reasonable strain level at failure.

Some of the main features of the HS model are stress-dependent moduli according to a power law, (input parameter m); plastic straining due to primary deviatoric loading (input parameter E_{50}^{ref}); plastic straining due to primary compression (input parameter E_{oed}^{ref}); elastic unloading/reloading (input parameters E_{ur}^{ref}, ν_{ur}); failure according to the Mohr-Coulomb failure criterion (input parameters c', ϕ', ψ or s_u). The moduli vary along with effective stresses normalized with a reference pressure p_{ref} that is usually taken equal to 100 kPa.

3.3 NGI-ADP model

The NGI-ADP soil model (Grimstad et al., 2012) is a total stress elastoplastic model based on an anisotropic Tresca failure criterion. The anisotropic behaviour follows the ADP framework, where the s_u profiles for active (A), direct simple shear (D), and passive (P) stress paths are given directly as input parameters. The nonlinear hardening curves are described by the input of the peak s_u and corresponding shear strains (γ_A, γ_{DSS} and γ_P) in the three directions of shearing represented by triaxial compression, DSS, and triaxial extension. By interpolation between the three input curves, the model can represent the anisotropic behaviour of the clay for a general three-dimensional (3D) stress state.

Undrained triaxial and DSS tests are required to establish input parameters. The model can simulate an initial anisotropic consolidation stress state by means of the initial shear mobilization parameter τ_0/s_{uA}. The initial inclination of the stress-strain curves comes from the initial shear modulus G_0, which also defines the unloading/reloading modulus (G_{ur}) and the magnitude of the elastic strain components.

The model assumes that s_u varies linearly with depth within a soil layer. A constant s_{uAref} (reference active or triaxial compression) at a reference depth y_{ref}, together with a strength increase parameter s_{u_incA}, defines the s_u profile in the soil layer. Above y_{ref}, s_{uA} is constant and equal to s_{uAref}. The strength profiles for DSS and passive (triaxial extension) are defined from anisotropy ratios s_{uDSS}/s_{uA} and s_{uP}/s_{uA}.

4 CALIBRATION OF CONSTITUTIVE MODEL PARAMETERS

Constitutive model parameters are calibrated using the SoilTest tool in PLAXIS. Soil behaviour is assumed to be undrained. For Mohr-Coulomb and NGI-ADP, Undrained (C) option is selected. For Hardening Soil, Undrained (A) and Undrained (B) modes are studied, based on effective and total stress parameters, respectively. Parameters have been calibrated from both CIUC and DSS test results.

Figure 6 shows the Mohr-Coulomb, HS Undrained (B) and NGI-ADP model versus the DSS test results normalized with respect to the maximum shear stress in the test. Figure 7 shows the normalized CIUC tests versus the constitutive models. The constitutive models considered cannot capture strain-softening observed for some of the CIUC tests. The calibrated model parameters are meant to represent best estimate values according to engineering judgement. Figure 8 and Figure 9 show the Hardening Soil (Undrained A) model versus the DSS and CIUC results, respectively.

Tables 1 to 4 summarize the input parameters to the constitutive models for triaxial (TX) and DSS shearing modes. For the NGI-ADP model, the TX (both compression and extension) and DSS modes are accounted for by the input parameters. NGI-ADP parameters for TX extension and HS unloading/reloading modulus ratio E_u^{ref}/E_{50}^{ref} are assumed for Ristijärvi peat based on engineering judgement and typical observations for clays (e.g., D'Ignazio, 2016).

5 DISCUSSION

The calibrated soil model parameters from Section 4 reflect the anisotropic nature of peat. The estimated moduli values for all three constitutive models are approximately 1.5 times larger for TX compression compared to DSS.

The Mohr-Coulomb model does not seem to be suitable to represent the strain-hardening/strain-softening behaviour of peat. Nevertheless, it can be used for total stress problems where the evaluation of deformations is not relevant or required by the design. The rigidity index G/s_u (≈20-30) estimated for Ristijärvi peat, roughly corresponds to a G_{50}/s_u value for a clay with plasticity index >150 according to Termaat et al. (1985), which is in line with the organic nature of the soil. Nevertheless, the FE G/s_u ratio may be influenced by sample disturbance.

On the contrary, the Hardening Soil and the NGI-ADP models can capture the strain-hardening behaviour of Ristijärvi peat. In particular, HS Undrained (B) and NGI-ADP nearly provide the same stress-strain curve. For HS Undrained (B), the stress dependency of stiffness is no longer defined according to the effective stresses but following the input isotropic undrained shear strength to the model. Therefore, E_{50}^{ref} is solely a function of s_u. As for Mohr-Coulomb, HS and NGI-ADP show stiffness parameters suggesting a significantly softer behaviour than the typical Finnish clays (e.g. D'Ignazio 2016; D'Ignazio et al. 2017). Hence, in stability calculations of embankments, larger deformations are anticipated in peat if the same target safety factors for clay or silt are adopted in the design.

The HS Undrained (A) model was capable of qualitatively reproducing both DSS and CIUC test results (Figures 8 and 9). In such case, the undrained shear strength is based on the input friction angle. The tests were simulated assuming vertical consolidation stresses σ'_{vc} (DSS) or isotropic cell pressures in line with the actual tests. A value of ϕ'=35° appears to be suitable to capture the range of test results. For triaxial compression, a higher friction angle ϕ'=44° was used to reproduce one of the tests from a specimen taken below the embankment. The selected ϕ' values are in line with the literature (e.g., O'Kelly, 2017).

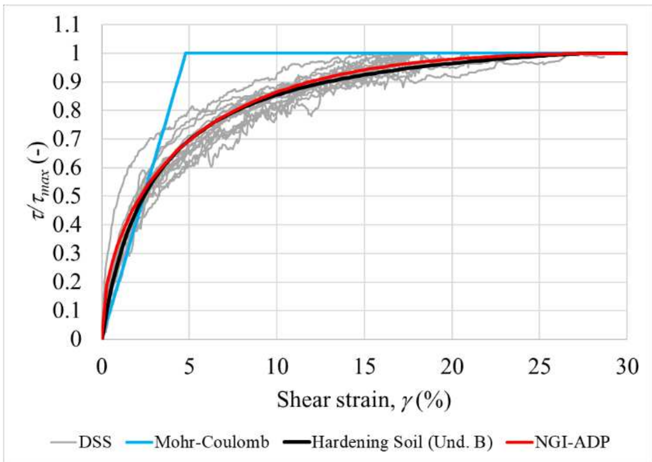

Figure 6. Normalized DSS tests on Ristijärvi peat vs Mohr-Coulomb, Hardening Soil (Undrained B) and NGI-ADP soil models.

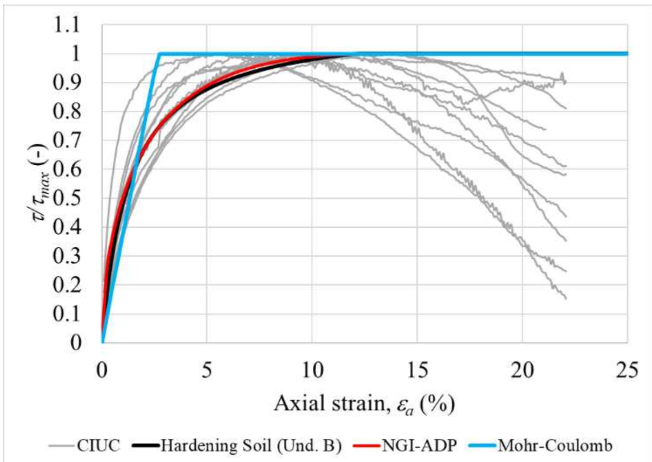

Figure 7. Normalized CIUC tests on Ristijärvi peat vs Mohr-Coulomb, Hardening Soil (Undrained B) and NGI-ADP soil models.

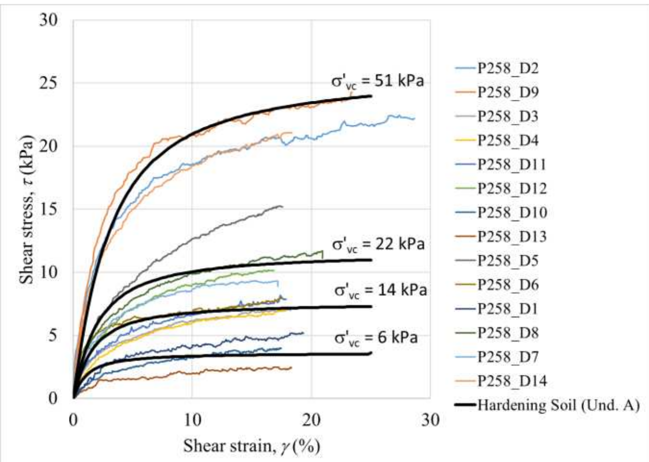

Figure 8. DSS tests on Ristijärvi peat vs Hardening Soil (Undrained A) model.

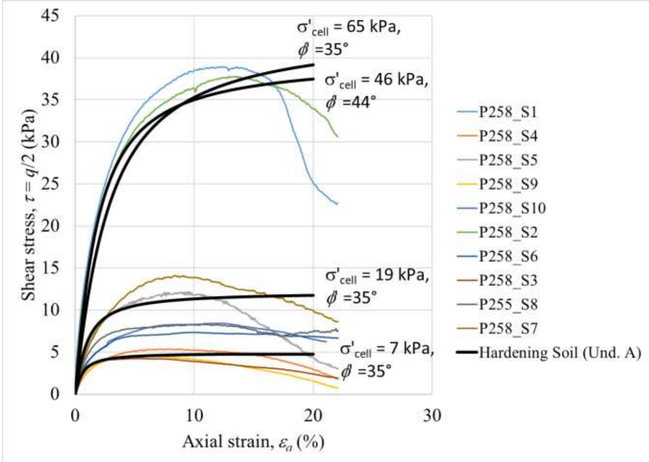

Figure 9. CIUC tests on Ristijärvi peat vs Hardening Soil (Undrained A) model.

Table 1. Input parameters for Mohr-Coulomb (Und. C).

Parameter	Unit	Test type	Value
G/s_u	-	TX	31.25
G/s_u	-	DSS	20.8
ν_u	-	-	0.495

Table 2. Input parameters for Hardening Soil (Und. A).

Parameter	Unit	Test type	Value
E_{50}^{ref}	kPa	TX	1800
E_{50}^{ref}	kPa	DSS	1200
$E_{ur}^{ref}/ E_{50}^{ref}$	-	TX, DSS	5*
m	-	TX, DSS	0.5
c'	kPa	TX, DSS	1
ϕ'	°	TX	35-44
ϕ'	°	DSS	35
ψ	°	TX, DSS	0
ν_{ur}	-	-	0.2

**Assumed based on engineering judgement*

Table 3. Input parameters for Hardening Soil (Und. B).

Parameter	Unit	Test type	Value
E_{50}^{ref}/s_u	-	TX	90
E_{50}^{ref}/s_u	-	DSS	60
$E_{ur}^{ref}/ E_{50}^{ref}$	-	TX, DSS	5*
ν_{ur}	-	-	0.2

**Assumed based on engineering judgement*

Table 4. Input parameters for NGI-ADP model (Und. C).

Parameter	Unit	Value
G_{ur}/s_u	-	700
τ_0/s_{uA}	-	0
γ_A	-	20
γ_{DSS}	-	30
γ_P	-	40*
s_{uDSS}/s_{uA}	-	0.75
s_{uP}/s_{uA}	-	0.5*
ν_u	-	0.495
K_0	-	1.0

**Assumed based on engineering judgement*

In practice, HS Undrained (A) model can be used to evaluate the stability of embankments on peat during e.g. staged construction or, in general, to take into account the stress history.

When selecting s_u from triaxial compression data for an elastoplastic strain-hardening soil model, the input s_u values should be lower than the peak values to avoid overestimation of the embankment failure load. For instance, if s_u for HS or NGI-ADP model is taken at 5% strain level from Figure 7, this would result in a shear mobilization level of ≈85%, i.e., would include a softening material factor of ≈1.15.

6 CONCLUSIONS

This study illustrated the calibration of the Mohr-Coulomb, Hardening Soil and NGI-ADP soil constitutive model parameters for Finnish peat from Ristijärvi, in the North of Finland. Model parameters are calibrated from undrained triaxial compression and direct simple shear tests on samples taken along an existing railway track, below the railway embankment, the counter berm, and the free field.

The model calibration revealed the anisotropic nature of peat, resulting in generally higher values of moduli in triaxial compression compared to direct simple shear. Moreover, the calibrated parameters indicate a notably softer behaviour compared to Finnish soft clays. This aspect requires further attention regarding the embankment stability calculations and design safety factors, as larger deformations will occur in peat if the same safety design requirements for clay or silt are adopted. Further, the selection of undrained shear strength for stability analyses shall account for the strain-softening behaviour observed from triaxial test results.

The FE parameters presented in this study are meant to be used for preliminary undrained stability and serviceability calculations of road/railway embankments on Finnish peat. Detailed design shall be supplied with site-specific data and the FE models must be recalibrated.

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