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Engineering properties of Norwegian peat for calculation of settlements

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ABSTRACT

Despite some 9% of Norway's area being underlain by peat the geotechnical characteristics of the material have not been well documented. As peatlands form an excellent carbon sink there is much pressure on planning authorities to avoid the excavation of peat for infrastructure development. In engineering projects in peat often the resulting settlements both in the short and long term are of greatest concern. Because of the high variability of peat, several, largely empirically based, methods exist for predicting settlement on peat. These were often developed based on specific cases. These techniques have shown to be inadequate to generically predict long term creep settlements and the development of settlement with time. Here the engineering characteristics of peat from several sites in the Trondheim area of mid-Norway are studied using a series of field and laboratory tests including three types of oedometer test. The laboratory and field data, together with empirical correlations found in the literature, were used to provide input into the commercially available constitutive model Soft Soil Creep (SSC) in the computer code PLAXIS. The model was initially calibrated using the laboratory test results and then applied to the back-analysis of two full scale field trials in the Trondheim area, where the peat properties were significantly different. The modelling showed that SSC captured well the vertical settlement versus time behaviour of the peat. Guidance is provided for selecting the critical input parameters for SSC such as stiffness, yield stress and permeability. This work contributes towards efforts being made to bridge the gap between the numerical modelling community and practicing engineers.

1. Introduction

Peat is frequently found in the high latitudes of the Northern Hemisphere. Canada and Russia have the most extensive areas of peatlands although extensive areas are found in northern Europe, especially Finland, Sweden, Norway, Ireland, and the Netherlands. The geotechnical characteristics of peat from some of these countries have been well documented, for example by Landva (2007), Hendry et al. (2012), Sarkar and Sadrekarami (2020) Helenelund (1980), Carlsten (2000), Long and Boylan (2013), Den Haan et al. (1995), Den Haan and Kruse (2007), Den Haan and Feddema (2013), Zwanenburg and Jardine (2015) and Murano and Jommi (2021).

However despite there being some 28,300 km^2 of peat in Norway, representing 9% of the country's area (NIBIO, 2016), the geotechnical characteristics of Norwegian peat have not been well documented. In the past, where peat was encountered, it was simply excavated and replaced by imported fill material. In addition the impact on climate change of peatlands was not as well understood. It is estimated that 10,000 m^2 of peat can store about 5000 tons of carbon (NPRA, 2015; Oksholen, 2020). Norwegian peat areas contain the carbon equivalent to Norway's total climate emissions for 66 years (SABIMA, 2015). As a result planning authorities have adopted new regulations prohibiting the cultivation of peat areas (Norsk-Lovtidend, 2020) and are reluctant to permit developments which involve the excavation and removal of peat (Menon-Economics, 2017; Miljødirektoratet, 2018; Miljødirektoratet, 2020; NPRA, 2015).

Trøndelag county in mid Norway is the largest peat region in Norway, where peat covers some 18% of its surface area. Peat samples were obtained from seven sites in the Trondheim area of Trøndelag and samples from six of these sites were subjected to a comprehensive series of 1D oedometer tests. The resulting geotechnical engineering properties are detailed and compared with those from other peat soils found

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worldwide.

One of the main geotechnical challenges with development on peat areas involves reliable prediction of both short and long-term creep type settlements. Existing empirical or analytical models designed for clay soils are known to be unreliable in peat, especially for cases where there are several loading stages, where the consolidation time needs to be estimated and in particular for the calculation of long-term settlements.

The need to better understand the geotechnical properties of peat, such as its yield stress, 1D compressibility and permeability, to permit development on these areas without removal of the peat is the focus of this work. Use of the widely available Soft Soil (SS) and Soft Soil Creep (SSC) models in the finite element software PLAXIS (PLAXIS, 2020) are explored using input data derived from the oedometer tests and empirical correlations published in the literature. The SS and SSC modelling is tested on the measured data from some full-scale field loading trials at two sites in Trøndelag. Guidance for future testing and modelling of peat soil for the purposes of predicting short and long-term settlements is provided.

2. Previous geotechnical studies of Norwegian peat

Early work on the geotechnical properties of Norwegian peat involved studies on the applicability of peat as an impervious material for earth dams, with a focus on peat permeability (Tveiten, 1956). In the 1960's the Norwegian Public Roads Administration (Statens vegvesen) undertook some studies on the 1D compression of peat and Flaate (1968) published some empirical charts to permit the estimation of peat settlement due to the construction of road embankments. Later the same organisation reported on the use of lightweight fills, in the form of expanded polystyrene (EPS) blocks, to construct roads across peatlands

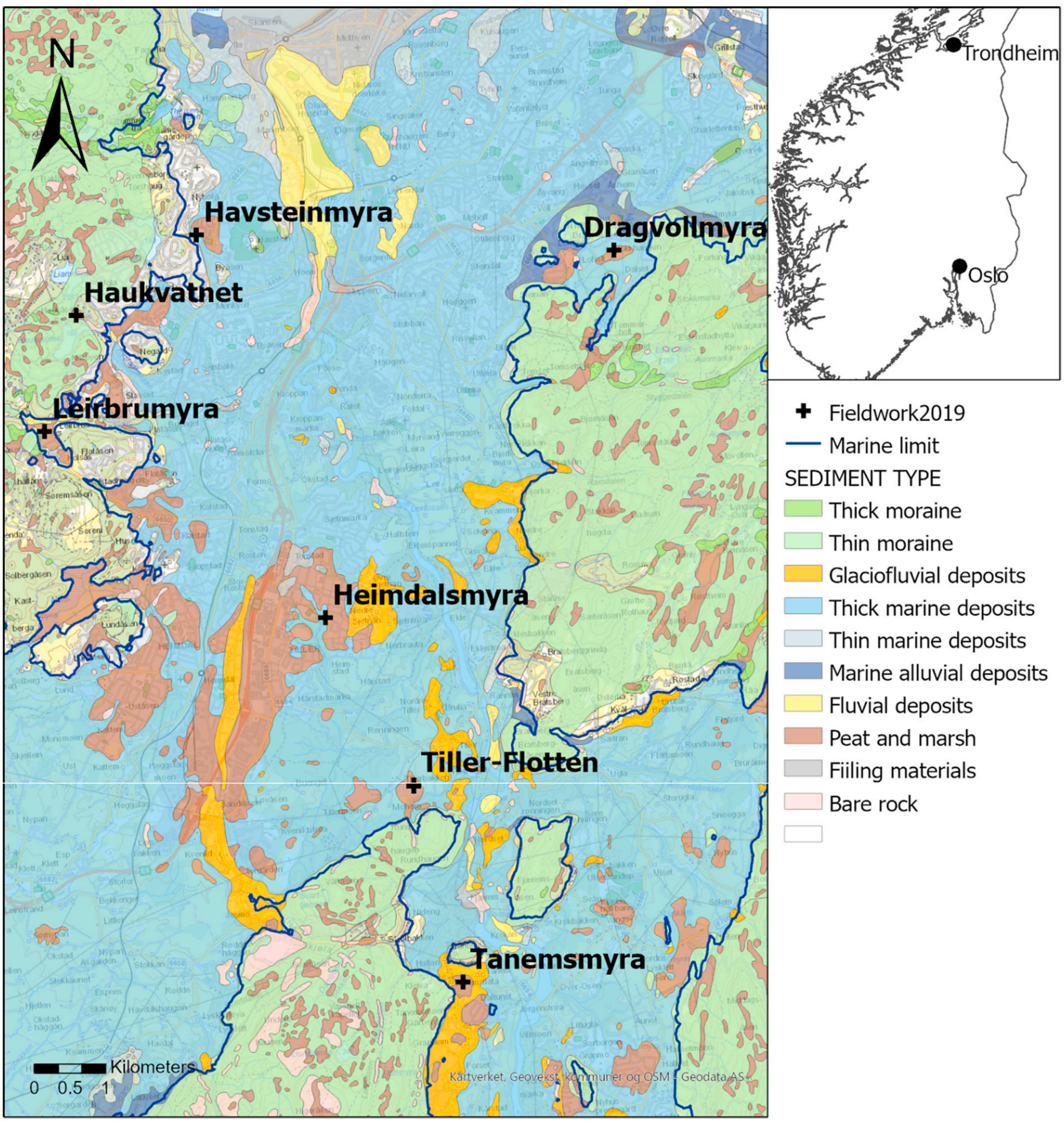

Fig. 1. Location of test sites superimposed on Quaternary geological map of the area. Note the blue line indicates the marine limit. Base map from Norwegian Geological Survey (NGU) http://geo.ngu.no/kart/losmasse/ (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

(Frydenlund and Aabøe, 1989; Østlid, 1987). Kjærnsli (1989) used Flaate's data together with some additional measurements to update the empirical charts for estimating settlement of road embankments on peat.

Gautschi (1967) reported on some trial embankments and testing of peat at sites near Kongsvinger (south east Norway) and Oslo. Helenelund et al. (1967) also report briefly on these trials. Janbu (1970) reports on some oedometer tests carried out on peat samples from a site in Steinanmyra in Trondheim, which is close to the sites under study here. Janbu (1970) showed that the behaviour of the peat was consistent with that of a wide variety of other materials using his tangent modulus approach. Hove (1972) carried out some measurements on the properties of peat and the settlement of noise protection bunds constructed at Heimdalsmyra in Trondheim. This is one of the sites studied here. Long and Boylan (2013) also reported on some of the data from this site. Herje (1978) summarised the experiences of building houses on peat areas in Trondheim during the years of 1972–1975. Flaate (1989) summarised some of the main geotechnical properties for peat when being used as a base sealer beneath landfills.

More recent work on Norwegian peat has included a study of the effects of a tunnel leakage on the overlying peatlands (Kværner and Snilsberg, 2008), the development of a 5 km temporary construction site road over a 3 m thick layer of peat (Øiseth and Sleipnes, 2008), the use of airborne electromagnetic technology to map peat thickness (Silvestri et al., 2019) and the development of some sophisticated numerical models to study creep in peat (Boumezerane and Grimstad, 2015; Grimstad et al., 2017).

3. Study sites

Paniagua et al. (2021) present data for nineteen Norwegian peat sites where index data was available. Here seven of these sites in the Trondheim city area were chosen. Sampling, in situ testing, laboratory oedometer testing and shear wave velocity measurements were carried out during Summer 2019. The location of the sites, superimposed on the Norwegian Geological Survey map of the area, is shown in Fig. 1. A summary of the sites is also presented on Table 1. Six of the seven sites are located under the marine limit which is the highest former sea level in the area after disappearance of the ice. It means that the peat deposits under the marine limit were formed over a saline environment, and usually marine clays are found beneath. Only the Haukvatnet site is at a slightly greater elevation.

Table 1
Summary of study sites.

Site	Description	Related references and details
Tanemsmyra	The site is in Klæbu. It has a relatively flat topography. Field investigations show significant depths of peat from 2.5 m to 7.0 m within the area. Clay, sometimes quick, underlies the peat.	The area has recently been under study due to a road construction. A roundabout, parking areas and bus stop have been built on an area underlain by peat. A peat landslide occurred during excavation trials (Berbar, 2019)
Tiller-Flotten	The site is in Tiller. The research site is relatively flat. The area is dominated by peat over a thick deposit of marine clay (quick from 7 m below terrain). The peat thickness is between 1.8 m and 2.5 m.	This site is also the Norwegian Geotechnical Test Site (NGTS) quick clay site (L'Heureux et al., 2019)
Heimdalsmyra	It is in Tiller. The main layering in the area consists of 2 m to 4.2 m of peat over either a medium or stiff clay. Large and intense drainage measurements were carried out during the, 1970's that caused settlements in the peat layer.	Characterisation of the peat and behaviour of some trial fills are described by Hove (1972) and Long and Boylan (2013)
Leirbrumyra	It is in the ski arena at Granåsen. The site has a relatively flat topography. The peat layer has a variable thickness up to 11 m thick over variable moraine.	The area has been under study for some time due to the gradual development of the ski arena, e. g. for the World Cup in 1997. There has been a progressive settlement of the area due to filling by gravel to make up car parks and hard standings
Dragvollmyra	The site is in Dragvoll. It has a relatively flat topography. The area is dominated by peat of on average 5 m thickness over quick clay. Bedrock is found under the quick clay.	The quick clay at the site is described by Emdal et al. (2012)
Havstein	The site is in Byåsen. The peat thickness varies between 2 m and 4 m.	The site has a relatively variable topography mainly due to construction of some sports facilities and a pedestrian path in recent years (Trondheim-Municipality, 2019).
Haukvatnet	It is in Ugla. The site has a relatively flat topography. The site over the marine limit. Peat varies between 2 m and 6 m thickness. Moraine with a layer of coarse material is found beneath the peat. Bedrock is estimated to be shallow.	Part of the site is planned to become a parking area (Trondheim-Municipality, 1970). However no construction has been done so far.

4. Methodology

4.1. Sampling

A peat sampler from Eijkelkamp was used to obtain a 52 mm diameter "half core" of the peat over the full depth profile. This sampler is similar to the Jowsey auger (Jowsey, 1966). The peat was logged on site using the extended version of the von Post and Granlund peat classification system (Hobbs, 1986). Samples were taken for water content measurements and these were used later in conjunction with field shear wave velocity measurements (V_s). A photograph of a typical sample of Trondheim peat is shown in Fig. S1 of the Supplementary Data section of the paper.

In order to minimise sample disturbance effects, block samples of the peat from all sites, except Havstein, were obtained for laboratory compressibility testing by hand carving the material using a knife with a serrated cutting edge from the bottom of a hand excavated hole at between 0.5 m and 0.6 m depth and below the water table. The sampling procedures followed those recommend by ISSMFE (1981). Samples were transported to either the laboratories of the Norwegian University for Science and Technology (NTNU), Trondheim or to University College Dublin (UCD) in Ireland.

4.2. In situ shear wave velocity measurement

Shear wave velocity (V_s) measurements are extremely valuable in site characterisation studies (e.g. distinguishing peat from gytta and soft clay), assessing sample disturbance effects, yielding parameters for numerical modelling (e.g. yield stress, p_{vy}' and the small strain shear modulus, G_{max}) and for use in making estimates of settlement (Long, 2022). Here V_s measurements were made in the field using the portable downhole sonde which was developed for the purposes of taking V_s readings through a vertical peat column by Trafford and Long (2020). A broad bandwidth geophone (10 Hz resonant frequency) is encompassed within a high-density polyethylene housing to form the sonde. The sonde was initially pushed to the bottom of the peat profile, down the same hole which was used for peat logging and water content sampling. The V_s readings were taken at the same depth intervals and frequency and down the same borehole as that used for logging the peat. Measurements were made as the sonde was retrieved at intervals of 0.1 m to create a continuous seismic profile through the full peat column. The transmitted shear wave was produced at the surface by striking a hammer against an instrumented block.

4.3. Index testing

Water content (oven dried at 80 °C for 24–48 h) and density measurements were carried out to the requirements of BS1377 (BSI, 1990a). The water content measurements were made at NTNU within 24 h of sampling. Organic content was obtained by determination of the loss on ignition at 440 °C for five hours (Arman, 1971).

4.4. Laboratory oedometer testing

A summary of the oedometer testing carried out is given on Table 2. Three different test types were used. Incrementally loaded (IL) oedometer tests in accordance with BS1377 (BSI, 1990b) were carried out at UCD. The specimens were 76 mm in diameter and 19 mm high (H_0) Each load was maintained for a period of 24 h as required by the British Standard.

At NTNU, incrementally loaded oedometer tests were carried out using the Janbu torvødometer (peat oedometer) (Janbu, 1970). The apparatus has been also tested by Long and Boylan (2013). The sample is 54 mm in diameter and 50 mm in height. The motivation for using this

Table 2
Summary of IL, torvødometer and CRS tests.

Site	Test No.[1]	Strain rate (%/hr.)	w_i (%)	ρ (kg/m^3)	LOI (%)	e_0	p_{vy}' (kPa)	M_0 (MPa)	M_L (MPa)	m	C_c	C_{sec}	$C_α/C_c$	k_{v0} (m/s)	β
IL tests															
Tiller-Flotten	ML-2	–	903	1.05	90	12.88	9.0	0.11	0.05	5	5.83	0.025	0.060	7.0×10^{-9}	3.0
	ML-5	–	834	1.05	90	11.90	10.8	0.13	0.06	4	6.45	0.033	0.066	1.0×10^{-8}	3.5
Leirbrumyra	ML-3	–	711	1.05	95	9.88	8.5	0.15	0.09	6	3.81	0.022	0.063	5.0×10^{-9}	3.5
	ML-4	–	751	1.05	95	10.43	13.5	0.37	0.07	5	4.57	0.023	0.058	1.0×10^{-8}	3.75
	ML-6	–	721	1.05	95	10.02	10.3	0.24	0.08	4	4.74	0.026	0.060	1.5×10^{-8}	3.5
Haukvatnet	ML-7	–	683	1.05	86	9.95	12.8	0.22	0.08	3	4.93	0.025	0.056	1.0×10^{-8}	3.0
	ML-8	–	771	1.05	86	11.24	12.0	0.19	0.07	5	6.12	0.024	0.048	1.6×10^{-8}	3.5
Torvømeter tests															
Tanemsmyra	TM-1	–	1459	1.05	98	20.01	7.3	–	0.03	3	11.97	0.016	0.028	–	–
	TM-2	–	1290	1.05	98	17.69	11.0	0.14	0.06	3	11.21	0.022	0.037	–	–
Tiller-Flotten	TFM-1	–	807	1.04	90	11.51	12.8	0.46	0.06	1.5	8.76	0.076	0.109	–	–
	TFM-2	–	960	1.03	90	13.69	10.2	0.37	0.09	2.5	9.55	0.056	0.086	–	–
Heimdalsmyra	HM-1	–	548	1.10	98	7.5	10.8	0.27	0.10	5	3.57	0.022	0.052	–	–
	HM-2	–	512	1.02	98	7.0	10.8	–	0.15	5	3.36	0.026	0.062	–	–
Leirbrumyra	LM-1	–	673	1.05	95	9.35	10.3	0.55	0.10	3.5	4.86	0.029	0.062	–	–
	LM-2	–	684	1.05	95	9.5	11.0	1.47	0.10	4	4.41	0.028	0.067	–	–
Dragvoll	DM-1	–	892	–	95	12.39	9.8	0.35	0.06	2	8.97	0.020	0.030	–	–
	DM-2	–	903	1.04	95	12.54	9.8	0.18	0.07	2.5	8.53	0.022	0.041	–	–
Haukvatnet	HkM-1	–	775	1.07	86	11.3	11.5	0.59	0.10	2	7.99	0.038	0.058	–	–
	HkM-2	–	916	1.03	86	13.35	11.5	0.89	0.09	2	8.61	0.028	0.047	–	–
CRS tests															
Tanemsmyra	TM-1	3	1382	1.03	98	18.90	6.5	–	0.05	4.5	9.95	–	–	1.0×10^{-7}	1.0
	TM-2	3	1629	0.99	98	22.28	5.0	0.18	0.04	4.5	13.97	–	–	–	–
Tiller-Flotten	TFM-1	3	680	0.96	90	9.70	7.3	0.13	0.06	5	4.28	–	–	4.0×10^{-8}	3.5
Heimdalsmyra	HM-1 Turn 90°	3	523	0.96	98	7.16	12.0	0.20	0.05	5	3.26	–	–	2.5×10^{-8}	3.0
	HM-2	5	517	0.98	98	7.07	10.3	0.24	0.05	6	3.23	–	–	1.0×10^{-7}	4.2
	HM-3 Turn 90°	10	517	0.99	98	7.07	9.8	0.23	0.07	5	2.82	–	–	1.5×10^{-7}	4.5
	HM-4	3	542	1.04	98	7.41	11.7	0.13	0.06	6	3.26	–	–	2.0×10^{-8}	5.5
Leirbrumyra	LM-1	3	662	1.02	90	9.20	8.0	–	0.08	5	4.08	–	–	4.0×10^{-8}	2.5
Dragvoll	DM-1	3	792	–	95	11.0	7	–	0.04	6	4.8	–	–	1.0×10^{-7}	4
Haukvatnet	HkM-1	3	846	1.03	86	12.33	9.5	0.20	0.05	5.5	5.6	–	–	1.0×10^{-7}	4

Notes: 1: all samples are from a depth of 0.5 m.
w_i: initial water content, ρ: density, LOI: loss on ignition, e_0: initial void ratio, p_{vy}': yield stress, M_0 / M_L: constrained modulus at in situ stress and at yield stress, m: modulus number, C_c: compression index, C_{sec}: creep coefficient related to strain, k_{v0}: soil permeability at in situ stress, β: rate of change of permeability with changing strain.

equipment was that the specimens are thicker than the 19 mm to 20 mm for standard oedometers. Using thicker oedometer samples of peat is also standard practice in Sweden (samples 100 mm in diameter and 45 mm high) and in Finland (samples 150 mm in diameter and 100 mm high), see Carlsten (2000) and Helenelund et al. (1972). The load increments and loading period were as for the IL tests.

Also at NTNU, continuous rate of strain (CRS) tests were carried out to investigate strain - rate effects. These tests were undertaken on 50 mm diameter by 20 mm high samples using the procedures outlined by Sandbækken et al. (1986). Three strain rates were initially used for Heimdalsmyra peat (3% / hour, 5% / hour and 10% / hour). Standard guidelines for CRS tests in Norway (NPRA, 2014) state that the strain rate must give pore pressures between 3% and 10% of the total vertical stress. The results of these tests are shown in Fig. S2 of the Supplementary Data section of this paper. The chosen rate of strain has little effect on the stress – strain and stiffness behaviour of the material. The two tests at the faster rates have higher permeability (k_v) and coefficient of consolidation (c_v) at stresses less than 15 kPa to 20 kPa. The results show that the samples which were tested at 90° to the direction they are orientated in the ground (horizontal direction), had the same properties as those tested at the vertical orientation. Based on these tests the chosen rate for the remaining tests was 3% / hour.

4.5. Laboratory shear wave velocity measurements

V_s measurements were made in the laboratory using a simple uniaxial loading cell. Samples were 70 mm in diameter and typically 50 mm in height. Bender elements, 3 mm long, were built into the top and bottom platen of the equipment. The samples were confined within a perspex cylinder lined with 2 mm holes to allow drainage during consolidation. Once at the required consolidation stress, high frequency shear waves (2 kHz to 5 kHz) were transmitted through the sample to determine the V_s value. First arrival time was obtained using the approach recommended by Lee and Santamarina (2005).

5. Laboratory test results

5.1. Description of peat

Visually the peat was similar at all sites and comprised a brown to dark brown fibrous peat with some wood fragments (see Fig. S1). An exception to this is that the peat at Tanemsmyra comprised an orange-brown fibrous sphagnum peat. At the sampling locations. The peat thickness varied between 1.8 m (Tiller-Flotten) and 6.9 m (Tanemsmyra), with an average of 4.3 m.

5.2. Typical index test results

Representative index test results for three of the sites are shown in Fig. 2. The three sites were chosen to represent that with highest water content (Tanemsmyra), lowest water content (Havstein) and a "typical" site (Heimdalsmyra). A summary of the average index properties for the peat on each site is given on Table 3.

Broadly speaking the peat at all seven sites is similar. The most significantly different site is that at Tanemsmyra which has the highest water content, a corresponding lowest shear wave velocity and is the only site where significant deposits of sphagnum peat were encountered. The sites at Dragvoll, Heimdal, Havstein and Tiller-Flotten have been influenced by land drainage measures in the past. This could have influenced the rate of natural decomposition of the peat and the current water content. The site at Havstein has the lowest water content and highest shear wave velocity. The von Post degree of humification (H), fine fibre content (F), coarse fibre content (R) and wood content (W) are similar for all sites. The sites at Leirbrumyra, Havstein and Haukvatnet have slightly higher H value. This may be associate with their similar geographical location, site orientation and geological background. These three sites are located at the western part of Trondheim, where the sunlight is more limited during the wintertime compared to the sites located towards the east (www.suncurves.com).

The peat at all the sites has very high organic content (loss on ignition - LOI) with only that at Havstein and Haukvatnet being less than 90%.

5.3. General stress – strain – time behaviour

A comparison between the results of two torvødometer tests and two incrementally loaded tests for the Tiller-Flotten peat (water content 807% - 960%) is shown in Fig. 3a and b. Here linear strain ($\varepsilon = \Delta H/H_0$, where H_0 = original sample thickness) versus root time curves for 5 kPa and 80 kPa applied stress are shown so a comparison can be made for

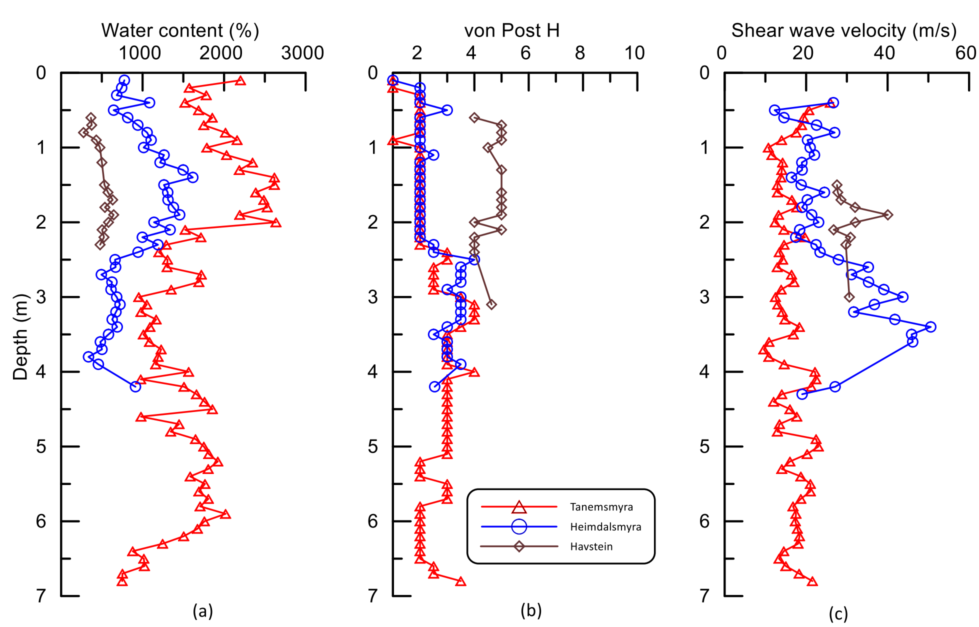

Fig. 2. Representative test results (a) water content, (b) von Post H and (c) shear wave velocity for sites with highest and lowest water content and "average" site.

Table 3
Summary of average index properties for peat (along the entire sampling depth) at sampling location.

Site	Peat thickness (m)	Water content w (%)	Loss of Ignition LOI (%)	Shear wave velocity V_s (m/s)	von Post scale parameters				
					H (−)		F (−)	R (−)	W (−)
Tanemsmyra	6.9	1612	98	16.2	H2-H3	2.5	1.3	1.1	1.0
Tiller-Flotten	1.8	831	90	23.6	H2-H3	2.6	1.9	1.0	1.3
Heimdalsmyra	4.2	913	98	27.1	H2-H3	2.5	1.8	1.4	0.0
Leirbrumyra BH1	5.6	679	95	20.0	H4	4.0	1.6	1.1	1.2
Leirbrumyra BH2	2.8	840	95	22.1	H3	2.9	2.0	1.0	1.0
Dragvoll	5.9	856	95	22.2	H3	3.2	1.9	1.3	1.2
Havstein	3.2	496	85	30.6	H4-H5	4.6	1.3	1.0	1.1
Haukvatnet	4.2	731	86	22.8	H4	3.8	2.0	1.2	1.2

Notes: H, F, R and W refer to degree of humification (1–10), fine fibre content (0–3), coarse fibre content (0–3) and wood content (0–3) respectively in the von Post scale.

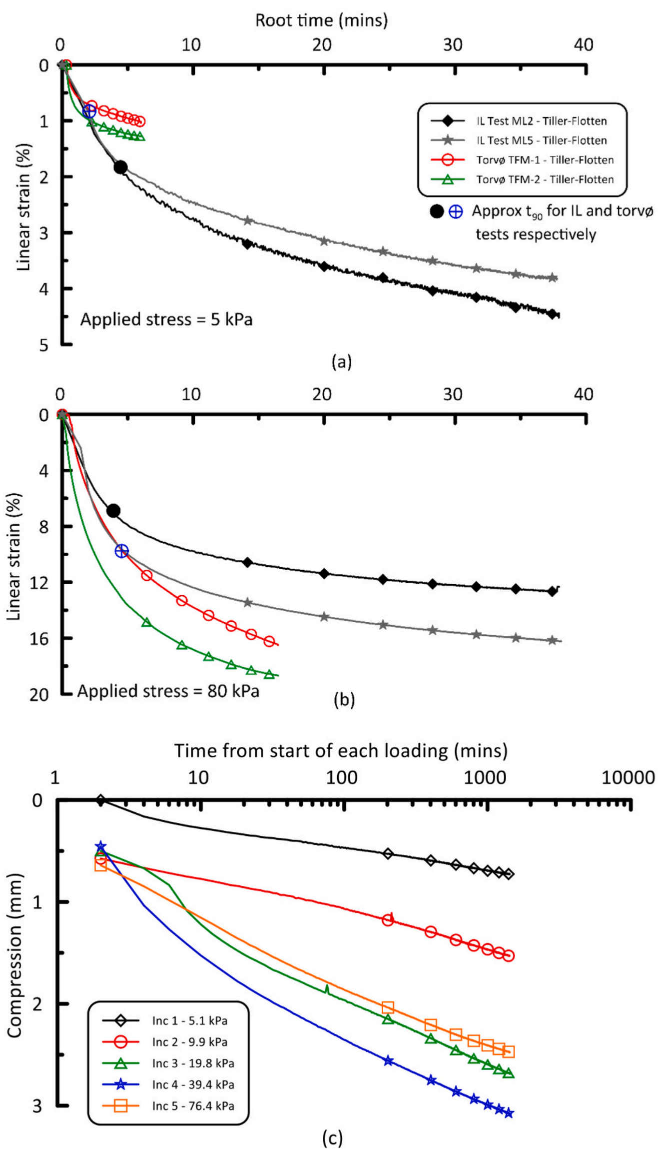

Fig. 3. Torvødometer and incrementally loaded oedometer tests for Tiller-Flotten peat (a) and (b) strain versus root time for applied stress 5 kPa and 80 kPa respectively and (c) all compression versus log time curves for IL test ML5.

stresses lower than and well above the yield stress. There is good consistency between each of the two test types. The difference between the IL tests and torvødometer tests is reflects the natural material variability of the peat. In addition to the differences in the resulting deformation between the IL and torvødometer tests, consolidation at low stresses took place much more rapidly in the torvødometer as can be seen by the location of the point corresponding to 90% consolidation (t_{90}) on the curves. At higher stresses the differences are not as pronounced. A similar finding was made by Samson and La Rochelle (1972) who showed that the difference between the field and laboratory data diminished as the load increased. Issues related to the effect of sample thickness on the rate of settlement of peat have been dealt with by Hobbs (1986) and others.

The compression versus log time plots for all five increments of a Tiller-Flotten IL test ML5 are shown in Fig. 3c. All the curves seem to be linear after approximately 20 min, which might give an indication of the onset of secondary compression in conventional soil mechanics terms. The creep rate is much lower for the 5.1 kPa and 9.9 kPa loading increments than for the remaining load increments. This suggests that after 9.9 kPa the yield stress of the sample has been exceeded. This will be discussed in more detail below.

Some typical IL, torvødometer and CRS test results, in this case for Tiller-Flotten peat, are shown in Fig. 4. The data are presented in (a) log stress (σ_v') versus linear strain ($\varepsilon = \Delta H/H_0$) format, (b) linear σ_v' versus ε, (c) constrained modulus M_t ($=\Delta\sigma_v'/\Delta\varepsilon$) versus σ_v', (d) creep coefficient C_{sec} ($\Delta\varepsilon/\Delta$logtime), (e) coefficient of permeability (k_v) versus σ_v' and finally (f) consolidation coefficient (c_v) versus σ_v'. In general, the results of the CRS and IL tests are very similar. An exception to this might be in the M_t values at stress greater than about 40 kPa. In this paper the comparisons are made between the derived parameters and correlations published in the literature made for stresses lower than 40 kPa since these are the operational stress in most practical cases.

The stress-strain, stiffness and creep behaviour of the peat in the torvødometer tests is very similar to that in the CRS and IL tests. However, the rate of consolidation, and therefore the resulting k_v and c_v values, are very different for the torvødometer tests. The torvødometer samples were prepared by trimming them to 54 mm diameter on a soil lathe as opposed to the technique of pushing a steel ring carefully into a block of peat to prepare the CRS and IL samples. It is possible that some leakage along the side of the torvødometer specimen resulted in the higher rate of pore pressure dissipation in these samples compared to the CRS and IL tests. Therefore in the subsequent sections k_v and c_v values from the torvødometer tests are omitted from the discussion.

A comparison between the test results from this project and data published previously for Norwegian peat by Kjærnsli (1989) is shown in Fig. S3 of the Supplementary Data section of the paper. Kjærnsli obtained some of his data from a personal communication with Janbu and also from Flaate (1968). The results from this project fit well with previously published data.

6. Engineering properties of peat

All the engineering properties as derived from the various oedometer tests are summarised on Table 2. The parameters are discussed in the

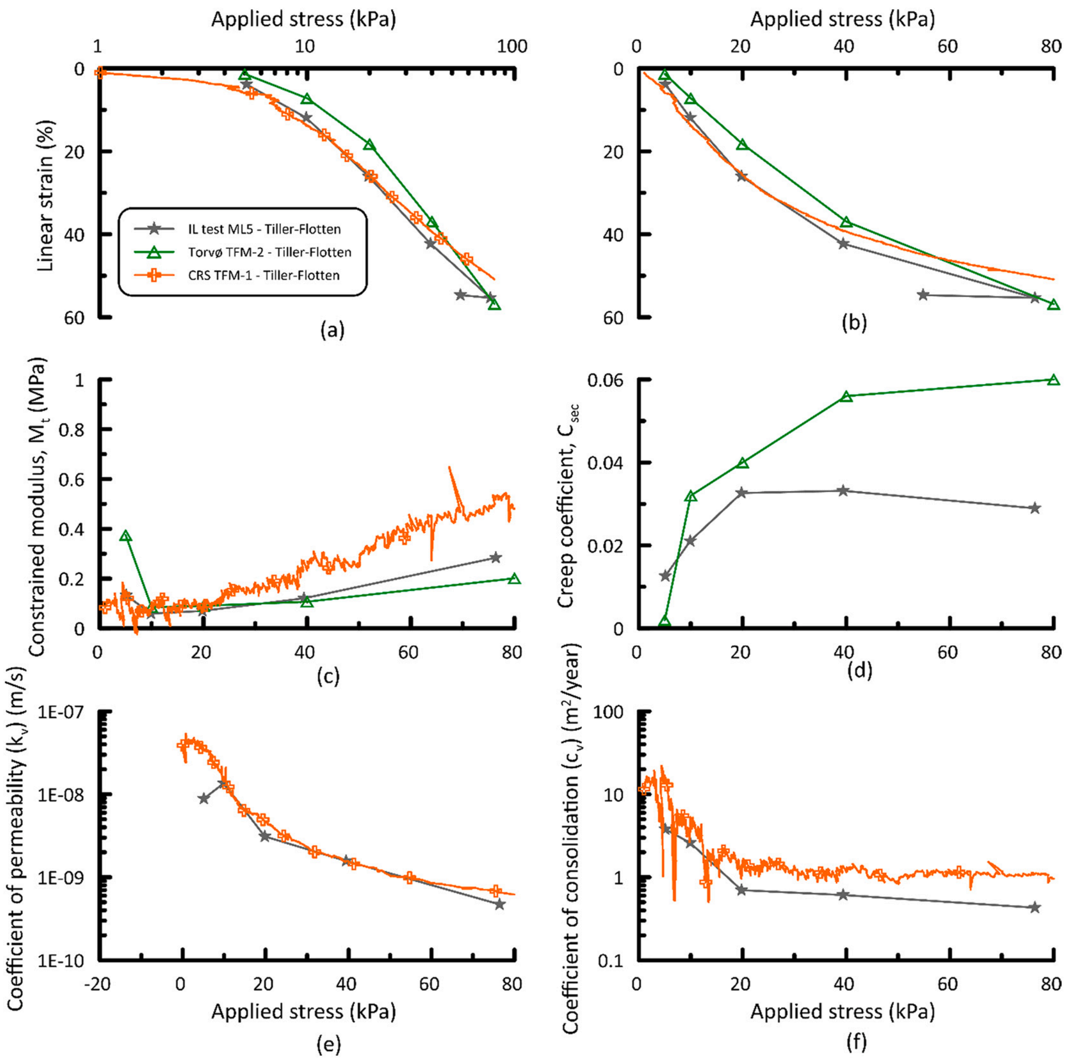

Fig. 4. Torvødometer, IL and CRS oedometer tests for Tiller-Flotten peat (a) log stress v linear strain, (b) stress v strain, (c) constrained modulus, M_t v stress, (d) creep coefficient, C_{sec} v stress, (e) coefficient of permeability, k_v versus stress and (f) coefficient of consolidation, c_v versus stress.

order in which they are obtained in oedometer testing, i.e. pre-yield stiffness first, followed by a discussion on the yield stress, post yield stiffness, creep and finally permeability and rate of consolidation.

6.1. *Constrained modulus at in situ stress (M_0) and at yield stress (M_L)*

Some typical plots of constrained modulus (M_t) versus versus σ_v' are shown on Fig. 4c. In engineering design the specific individual values often used are the constrained modulus for loads below the yield stress (M_0) and those around the yield stress (M_L). M_0 values are very low and vary between 0.1 MPa and 0.9 MPa with an average of about 0.6 MPa and show a weak trend of decreasing M_0 with increasing w_i, see Fig. S4 in the Supplementary Data section of this paper. The values for the IL and CRS tests are similar and slightly less than those from the torvødometer. It is possible that the top cap of the torvødometer was not in perfect contact with the peat sample for the first loading increment. This needs further investigation. Similar M_0 results were obtained by Long and Boylan (2013) for some Norwegian and Irish peats and by Helenelund (1969) for Finnish peat.

M_L values are similar for all test types and show a clear pattern of decreasing M_L with increasing w_i, see Fig. S5 of the Supplementary Data section. Values decrease from about 0.08 MPa at 600% water content to 0.04 MPa at w_i equal to 1600% and are very similar to the data published by Carlsten (2000) for Swedish peat.

6.2. *Yield stress (p_{vy}')*

Given the importance of the determination of the preconsolidation / yield stress in the prediction of settlements on soft soils significant emphasis was placed on the determination of this parameter. Use of the term “preconsolidation stress” is considered inappropriate for peat as the process of material formation was not by normal sedimentation. Here the term “yield stress” (p_{vy}') is used to nominally divide elastic and plastic behaviour (Chandler et al., 2004; Vaughan et al., 1988). The reasons why peat shows a yield stress are complex. Lefebvre et al. (1984) observed such an yield stress in the field and attributed it to snow loading, drainage, water table fluctuations and the creep characteristics. Hobbs (1986) also suggested that the structure of the plants and the decaying process that takes place in the upper layers of peat contributes to this “critical pressure”. Yield stress will also depend on fibre content, degree of decomposition and sampling technique as well as the season of sampling (e.g. relative to the ground water table) (Petrone et al., 2008).

There is no well accepted method for determining p_{vy}' in peat. Here four methods have been used namely the classical Casagrande (1936) and Janbu (Janbu, 1963; Janbu, 1969) techniques, as well as the “work method” of Becker et al. (1987) and by examination of the relationship between V_s and consolidation stress (σ_{vc}'). Some typical V_s versus σ_{vc}' plots for Tiller-Flotten and Heimdalsmyra peats are shown in Fig. 5a. Prior to measuring V_s the samples had been consolidated and the

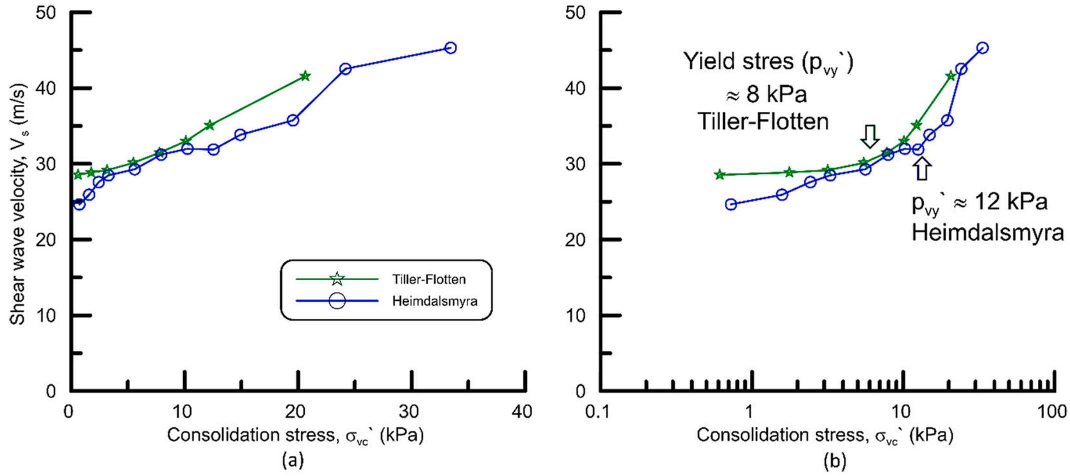

Fig. 5. Influence of consolidation stress on the measured V_s: (a) natural scale; and (b) logarithmic scale.

resulting strains and new thickness measured. The power function association between V_s and σ_{vc}' observed by others (Santamarina et al., 2001) is not seen here and the relationship between the two parameters is closer to linear. Yoon et al. (2011) have suggested that if the data are plotted in a semi-logarithm format, as shown in Fig. 5b, the yield stress can be obtained directly from the point of intersection of the two resulting straight lines. This suggests that p_{vy}' for Heimdalsmyra is slightly greater than that for Tiller-Flotten, consistent with the findings from the other techniques and the likely influence of the more extensive drainage at Heimdalsmyra.

In addition, an assessment of the creep rates at different stress, as for example shown in Fig. 3c, was used to estimate p_{vy}'. There were only small differences between the results using the different techniques and from the different test types. The Casagrande and Janbu methods gave the highest average p_{vy}' of about 11 kPa. All the others yielded an average of some 10 kPa. The average p_{vy}' obtained from the various methods are plotted against initial void ratio (e_0) in Fig. 6. Data for St. James Bay peat in Canada, which is similar to that under study here has been added to the figure (Ajlouni, 2000). Kogure and Ohira (1977) first suggested that p_{vy}' of peat was closely linked to initial void ratio e_0. Ajlouni (2000) re-examined their original relationship and suggested p_{vy}' (kPa) = $150/e_0$. The data measured here shows the same trend of decreasing p_{vy}' with increasing e_0 but the trend line is slightly below (lower p_{vy}`) than that proposed by Ajlouni (2000) with a trend line of p_{vy}' (kPa) = $110/e_0$ best fitting the data.

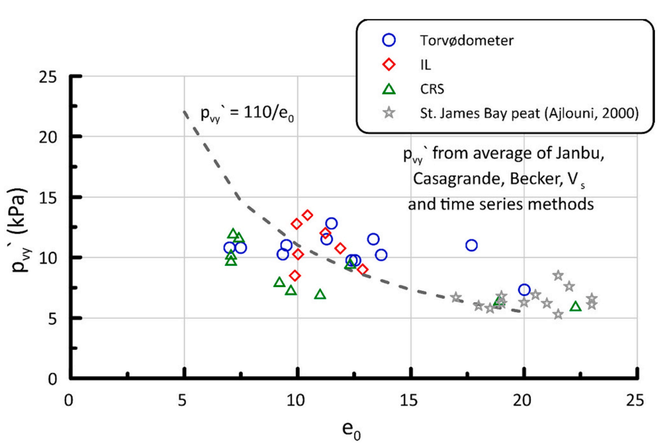

Fig. 6. Yield stress, p_{vy}' versus initial void ratio, e_0.

6.3. *Compression index (C_c) and modulus number (m)*

As can be seen from the data presented in Fig. 4a, the value of the compression index (C_c) has to be chosen with caution as the e (or ε) versus log σ_v' plot is non-linear. Here, C_c values have been selected for stresses ≈ p_{vy}' + 20 kPa as this is often the range of loading of most interest to practicing engineers. Values of C_c are plotted against initial water content in Fig. 7. As would be expected there is very good agreement between C_c and w_i. There are many correlations published in the literature on the relationship between C_c and w_i. Up to a w_i of about 800% the data fits well with the correlations proposed by Hobbs (1986) or Korhonen (1958). Beyond w_i of 800% the measured C_c values exceed those predicted by these two correlations. This finding is consistent with that of Kjærnsli (1989) who found Korhonen's correlation often formed the lower bound of measured data. However all of the test results fall within the linear relationship $C_c = w_i/100$ proposed by Mesri and Ajlouni (2007).

In Scandinavian practice settlement in the normally consolidated zone is often determined using the modulus number m, which is the slope of the M_t versus σ_v' curve after p_{vy}', see Fig. 4c. Values of m (again for stresses ≈ p_{vy}' + 20 kPa in the case of the CRS tests) are plotted against dry density (ρ_d) in Fig. S6 of the Supplementary Data section. Compared to soft clays and silts the m values are very low. The IL and CRS test values are in broad agreement with the values for Finnish clays published by Helenelund (1969), the trendline proposed by Carlsten (2000) for Swedish peat and also some values published by Janbu (1970) for tests from Steinanmyra in Trondheim. However the torvødometer values are lower than either of the two other sets and this may be an effect of the thicker sample in the torvødometer.

6.4. *Recompression index (C_r)*

Only the UCD IL tests included an unloading stage, which allowed the determination of recompression index C_r. The average C_r/C_c value was about 0.12 which is within the range 0.1 to 0.3 suggested by Mesri and Ajlouni (2007).

6.5. *Creep coefficient (C_{sec} or C_α)*

The creep coefficient, C_{sec} or C_α depending on its definition (C_{sec} = Δε/Δlogt and C_α = Δe/Δlogt), is often used by engineers predicting secondary compression in the field. As consolidation was relatively rapid sufficient data was generated in each load increment to reliably determine C_{sec}. Here it was always determined for time interval 100 mins to 1000 mins (see Fig. 3c) for consistency. C_{sec} values for all loading increments for a typical test are shown in Fig. 4d. Like C_c, all of the

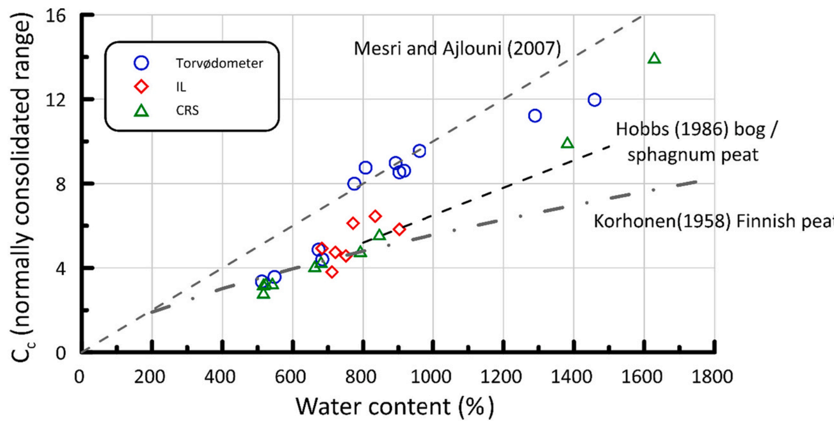

Fig. 7. Compression index C_c versus initial water content, w_i.

values for the load increment ≈ p_{vy}' + 20 kPa are plotted against initial water content in Fig. 8a. Both sets of tests show similar results. Most of the data is close to the mean value of about 0.025 reported by Carlsten (2000) for Swedish peats. The findings here are consistent with those of Hobbs (1986) who suggested C_{sec} was largely independent of water content, especially when w_i exceeds about 300%. The two exceptions are the results for Tiller-Flotten which show relatively high values of C_{sec}. The reason for this is not clear but the range of measured values is consistent with the limits 0.03 to 0.09 for peat suggested by Janbu Janbu (1970).

Although it does not directly give a prediction of in situ strain, many researchers and engineers prefer to use C_α (=Δe/Δlogt) rather than C_{sec} as it is used in the correlation between C_α and C_c, first introduced by Mesri and Godlewski (1977). This proposes that values of C_α/C_c are in the range 0.01 to 0.07 for all geotechnical materials. For peat, Mesri and Ajlouni (2007) suggest $C_\alpha/C_c = 0.06 \pm 0.01$. Mesri and Chen (1994) have pointed out that C_c and C_α need to be chosen consistently and to that end the values at ≈ p_{vy}' + 20 kPa have been selected here.

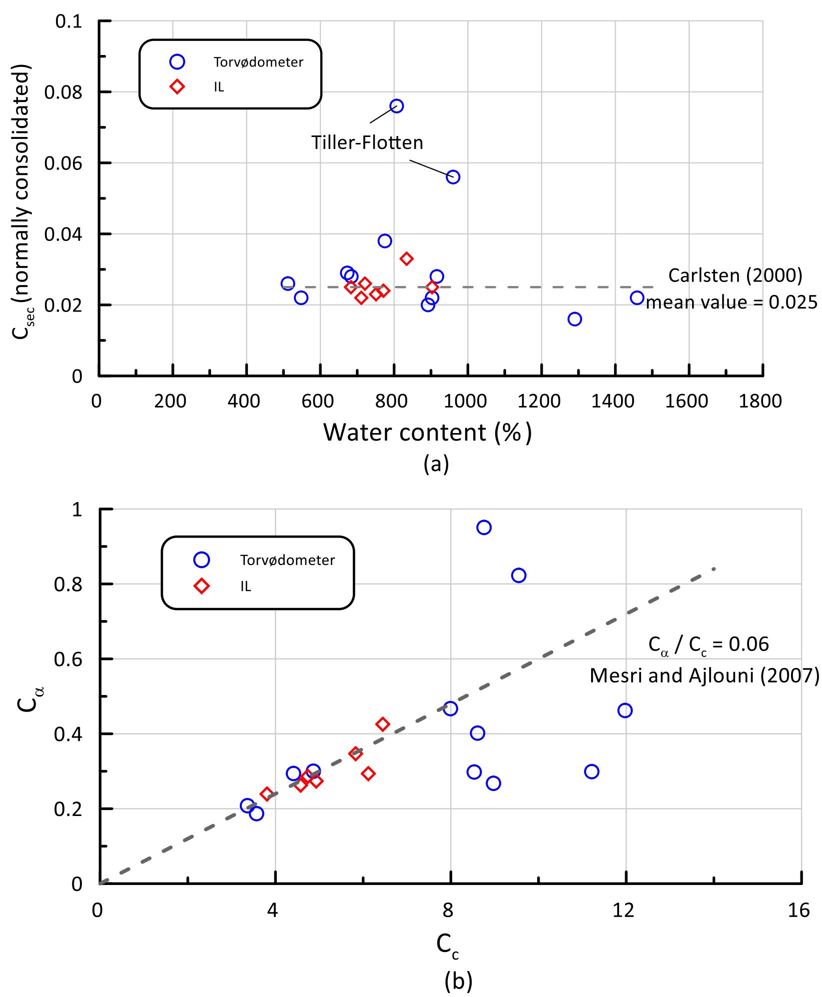

Fig. 8. (a) Creep coefficient, C_{sec} versus initial water content, w_i and (b) creep coefficient C_α versus C_c.

The data plotted in Fig. 8b follows the trend of C_α/C_c equal to 0.06 up to C_c of about 8. For values greater than 8 there is significant scatter around the 0.06 line with the two Tiller-Flotten tests showing very high values as discussed above.

6.6. Coefficient of permeability (k_v)

Peat is well known to display large values of permeability at in situ effective stress (k_{v0}) due to the open fabric and large void ratios. The great variability in the nature and arrangement of the peat fabric results in a wide range of permeability values (Ajlouni, 2000). In addition it is well accepted that the k_v value subsequently decreases rapidly with increasing stress. In this project k_{v0} values (i.e. at about 5 kPa) were determined from the c_v value obtained from the IL tests as well as the interpretation of the pore pressure data from the CRS tests by following the procedure outlined by Tavenas et al. (1983). The value of the pore pressure is taken to be two thirds that measured at the specimen base (Sandbaekken et al., 1986). The resulting values of are plotted against e_0 and w_i in Fig. 9 and Fig. S6 of the Supplementary Data section respectively. In Fig. 9 the measured data are compared to the empirical trendlines proposed by Mesri and Ajlouni (2007) and Kjærnsli (1989). These two sets of trendlines are more or less identical. The data fall within the two (albeit widely spaced) trendlines. The data for the CRS tests are one half to one order of magnitude greater than those from the IL. The differences are perhaps at least partly due to the different interpretation methods involved. This finding is consistent with that of Ajlouni (2000) who found CRS tests on James Bay peat yielded values 1 to 10 times greater than those obtained from the other measurements.

The same data are plotted against w_i in Fig. S6 of the Supplementary Data section. There is no clear trend in k_{v0} with w_i. However the data fall close to the trend lines suggested by Carlsten (2000) and Kjærnsli (1989).

The slope of the strain to log coefficient of permeability relationship is denoted by β and is defined:

$$\beta = \frac{\Delta logk}{\Delta \varepsilon} \tag{1}$$

β values are plotted against w_i in Fig. S8 of the Supplementary Data section. There is a faint relationship of decreasing β with increasing w_i as has been found by others but the trend falls below that of Carlsten (2000) for example. However the data fits very well with the range of values suggested by Helenelund (1969).

6.7. Poisson's ratio (ν)

Values for Poisson's ratio (ν) for peat and organic soils tend to be low, perhaps close to zero in many cases and in the range 0 ± 0.2. For example Den Haan and Kruse (2007) found ν increased with increasing density from values less than 0.2 for peat with a density of 1000 kg/m^3 to about 0.25 for organic soil with a density of about 1400 kg/m^3.

7. On settlement calculations in peat

7.1. Brief summary of settlement calculations in peat

Internationally, several theories specific for the prediction of settlement on peat have been developed based on simple empirical and semi-empirical correlations e.g. Flaate (1968), Al-Khafaji and Andersland (1981), Edil and Mochtar (1984), Edil and Simon-Gilles (1986), Stinnette (1998), Den Haan and El Amir (1994), Carlsten (2000), Sas et al. (2011), Szymanski et al. (2005), Noto (1987), Hayashi et al. (2016). It is difficult to make general statements on the success or otherwise of these methods for predicting settlement in peat. They are usually developed based on local correlations and with few parameters that vary with the method selected. These methods are primarily intended to enable a quick and approximate estimations and usually calibrated based on specific measurements making it challenging for generalisation. However it seems that primary settlement is often relatively well predicted but that secondary compression or creep is often underestimated (Lefebvre et al., 1984; Nichols et al., 1989) and that it is difficult to correctly predict the development of settlement with time.

7.2. Numerical calculation method adopted in this work

The various simplified methods, mentioned in the previous section, are not rigorous enough to handle the important aspects of settlement during both primary and secondary consolidation phases. They usually lack a through theoretical framework, but this emanates from the fact that they are primarily intended to be used for approximate first-hand estimations. While this is useful aspect of the simplified methods, there might arise instances where a more rigorous analysis was required, e.g. involving multiple loading sequences and creep deformations. In this work an attempt is made to address some of these issues. For this purpose the focus here is on the Soft Soil Creep (SSC) model available within the finite element program PLAXIS (2020). The SSC model (Grimstad and Nordal, 2018; Neher et al., 2001; Stolle et al., 1999; The et al., 1998; Vermeer and Neher, 1999; Vermeer et al., 1998) uses the isotache principle to deal with compression and the CamClay for shear

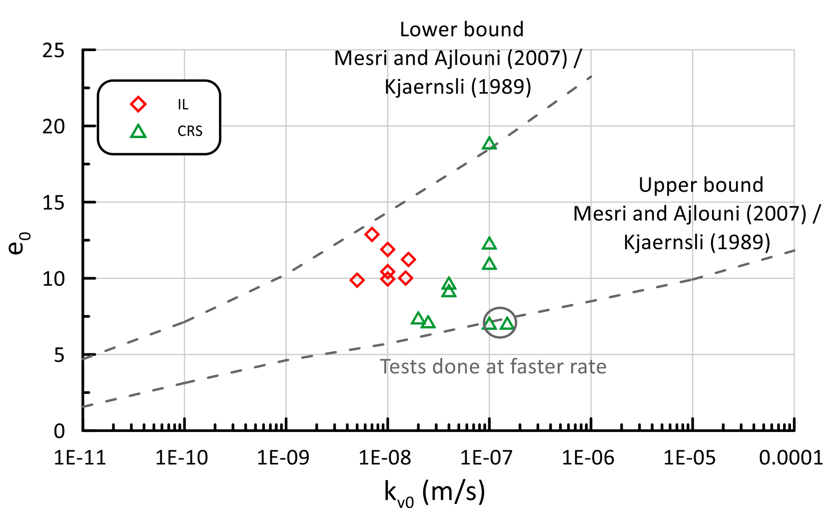

Fig. 9. Coefficient of permeability at in situ stress, k_{v0}, versus initial void ratio, e_0.

strength. It is designed to deal well with settlement due to stress changes, creep, ageing (creep hardening), stress relaxation and critical state shear deformations. The isotache concept (Ŝuklje, 1957) adopted in SSC enables modelling of the unique stress-strain-strain rate relationship through the entire consolidation phases. With this framework, the resulting yield stress is dependent on strain rate and this is a key aspect in modelling of soft clays and peats. For a complete and thorough description of the SSC model, as well as the background isotache theory, the reader is referred to the articles mentioned above and to the PLAXIS manuals (PLAXIS, 2020).

It is recognized that SSC is limited by assumptions such as ignoring anisotropy and/or fabric which are important in peat. It is also acknowledged that more advanced models exist within the research community that attempt to take into account such features, see for example Teunissen and Zwanenburg (2015), Boumezerane et al. (2015) or Grimstad et al. (2017). However these advanced models require knowledge on the material behaviour that requires field and laboratory tests well beyond standard engineering practice. The authors believe that SSC is a model that can give the most reliable predictions with input based on the results of standard geotechnical tests and empirically derived parameters as discussed in Section 6.

Results from two full-scale field cases are used to illustrate on how to overcome some of the limitations of the SSC model as well as on how to pick parameters to give an acceptable agreement between measured and simulated behaviour.

8. Description of full-scale loading tests and backanalysis

In this section a brief description of two field cases in the Trøndelag area is presented followed by the practical use of the established correlations for peat parameters with the SSC model. In both cases monitoring data is available with which to compare the numerical predictions.

8.1. The Selva Agdenes case study

This project involved the construction / upgrading of a standard two-lane road between the villages of Selva and Agdenes approximately 45 km north-west of Trondheim. The road is constructed over a mountainous area comprised of areas of bare rock interspersed with deposits of peat. In the area under study the peat thickness varies between 0.4 m and 2.0 m. Measured settlements after 5 months varied between 9 cm and 60 cm and increased approximately linearly with peat thickness. The cross section chosen for detailed analysis was at Profile 250 (Fig. 10a), where the peat is 2.0 m thick and the embankment height was 2.4 m.

The peat properties in the study area are shown in Fig. 10b. At Profile 250 w_i is relatively low and varied between 300% and 500%, with an average of 355% and von Post H was 5 indicating the peat is moderately decomposed. The ground water table was measured to be at 0.95 m depth.

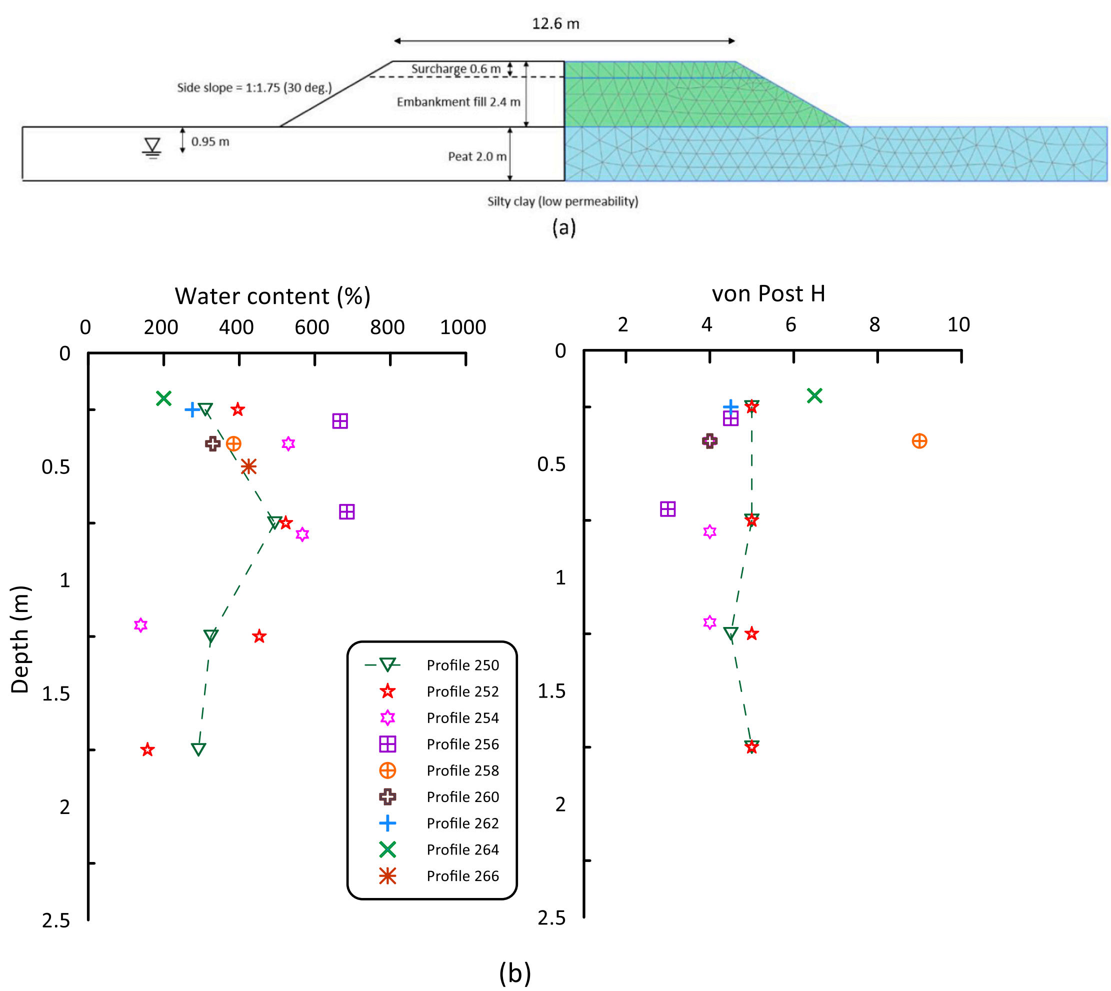

Fig. 10. Selva Agdenes (a) cross section at Profile 250 showing finite element mesh used in analyses and (b) peat properties.

8.2. The Heimdalsmyra case study

Some of the work at Heimdalsmyra has previously been reported by Hove (1972) and by Long and Boylan (2013). At this site full-scale loading trials included the monitoring of 2.5 m, 3 m and 4 m noise protection bunds, which were constructed using excavated peat from another area, and also of a 0.5 m thick gravel platform (to mimic site access roads and car parking) which was constructed directly on the peat.

Here the behaviour of the peat in some field compressometer tests (Janbu and Senneset, 1973) will be reported and modelled. The purpose of the tests was to provide input parameters for the design of the noise protection bunds, access roads and other hard standings. The set-up for these tests is shown in Fig. 11a. The 16 cm cast iron screw plate was initially screwed down to the required depth by hand. A hydraulic jack is connected to the outer pipe system which transmits the external forces during installation and the reaction forces during load testing. A series of inner pipes are connected to the screw plate for the purposes of load application. The inner and outer pipes are connected during installation. On reaching the required depth the connection between the two sets of pipes is released. The reaction force during loading is mainly taken by the anchor frame as shown in Fig. 11a, with settlement gauges connected to an independent anchored frame.

Here the focus is on a test conducted at Station 5 at a depth of 1 m. Three load steps were applied up to a maximum stress of 30 kPa. Each load step lasted 120 min. This test has been chosen for analysis as the loading involved steps less than and well above p_{vy}'.

The peat at Station 5 in Heimdalsmyra is very different from that at Selva Agdenes (Fig. 11b to 11d). The peat is about 2.1 m thick, has w_i of between 650% and 1200% with an average of about 900% and von Post H of 7 to 8, indicating a strong to very strong level of decomposition. The average density of the peat is about 1050 kg/m^3. The water table is close to ground surface.

8.3. Input parameters for SSC model

A summary of the input parameters for the peat for use in the SSC modelling at both sites is given on Table 4. Some notes on the selection of the parameters are as follows:

- o e_0 is the initial void ratio $= w_i{*}G_s$, where G_s the specific gravity is taken to = 1.5 as LOI exceeds 90% (Den Haan and Kruse, 2007).
- o γ is the unit weight.
- o λ^* is the modified compression index $= \frac{C_c}{2.3(1+e_0)}$, see Fig. 7.
- o κ^* is the modified recompression index $\approx \frac{2C_r}{2.3(1+e_0)}$, see Section 6.4, for volumetric compression.
- o μ^* is the modified creep index $= \frac{C_\alpha}{2.3(1+e_0)}$, see Fig. 8.
- o ν is Poisson's ratio, see Section 6.7. Generally low values were chosen here to ensure the highest possible elastic shear stiffness to predict realistic horizontal deformation. In doing so a relatively higher value of ν is used for Selva Agdenes due to a significantly lower water content and a lower degree of decomposition as compared to Heimdalsmyra. The Poisson's ratio adopted for Heimdalsmyra is 0.0 while for Selva Agdenes 0.15. These values are also consistent with the G_{max} interpreted from the shear wave velocity measurements. Some more details on the choice of appropriate values of ν for analyses such as these are given by Long et al. (2020).
- o POP is the pre-overburden pressure $= p_{vy}' - \sigma_{v0}'$, where σ_{v0}' is the in situ vertical effective stress and p_{vy}' can be obtained from Fig. 6. POP is an alternative to overconsolidation ratio (OCR), and both can be used in PLAXIS to specify the initial stress state (or initial size of the reference surface). However, for cases where the initial effective stress is very low POP is more convenient to use. A key factor for the use of the SSC model is the selection of appropriate parameters to ensure the correct initial state, including the initial creep rate (Waterman and Broere, 2005).
- o Waterman and Broere (2005) recommend, that in parallel with choosing OCR / POP, the user should also consider the creep ratio: $(\lambda^* - \kappa^*) / \mu^*$ which should typically fall within the range of 10 to 20 for most practical cases, with the highest values representing stiff soils which exhibit little creep. Here values of 12.5 and 13.8 are computed using the parameters in Tables 2, 3 and 4.
- o K_0^{NC} is the coefficient of lateral earth pressure in the normally consolidated condition. In this case K_0^{NC} values have been determined from empirical correlations published in the literature (Ajlouni, 2000; Den Haan and Feddema, 2013).
- o The strength of the material in the SSC model is limited to the lesser of the boundary of the cap surface (M parameter determined based on K_0^{NC}) and what is limited by the Mohr-Coulomb (MC) surface (defined by the friction angle, φ, and the dilatancy angle, ψ). These parameters need to be consistent with one another. In the SSC model M is calculated from:

$$M = 3\sqrt{\frac{\left(1-K_0^{nc}\right)^2}{\left(1+2K_0^{nc}\right)^2} + \frac{\left(1-K_0^{nc}\right)(1-2\nu)\left(\frac{\lambda^*}{\kappa^*}-1\right)}{\frac{\lambda^*}{\kappa^*}\left(1+2K_0^{nc}\right)(1-2\nu) - \left(1-K_0^{nc}\right)(1+\nu)}} \tag{2}$$

$$sin\varphi = \frac{3M}{6+M} \tag{3}$$

- o It is acknowledged that the chosen friction angle for peat is unusually high. Similar high values have been reported for peat from many other countries. Mesri and Ajlouni (2007) give a useful summary of international work on this topic. Den Haan and Kruse (2007) and others explain that the reinforcing effects of the mostly horizontally orientated fibers contributes to this effect and point out that the high frictional angles are mostly associated with low effective stress levels.
- o A tension capacity for the peat (σ_t = c - cotφ) can be applied based on cohesion (c) and friction angle. Here σ_t values of 2 kPa to 3 kPa have been used depending on the degree of decomposition (H) and water content of the peat (Dykes, 2008). These are assumed to be isotropic due to the limitations of the model.
- o k_{v0} and k_{h0} obtained from Figs. 9 and S7.
- o PLAXIS makes use of C_k, which is the permeability change index defined as:

$$C_k = \frac{\Delta e}{\Delta logk} = \frac{1+e_0}{\beta} \tag{4}$$

where β is obtained from Section 6.6 and Fig. S7 in the Supplementary Data section. The values chosen are consistent with the recommendation of Mesri and Ajlouni (2007) who suggested C_k/e_0 for peat is about 0.25.

8.4. Simulation of oedometer test

To complement the analysis of the full-scale field tests it was decided to initially simulate some oedometer test results, see Fig. 12. This work would also serve to check and calibrate the input parameters. These simulations are all one-point analyses, so pore pressure dissipation is not considered. The sample depth was assumed to be 1 m, the input parameters chosen were those for Selva Agdenes (Table 4) with the water table assumed to be at ground level as observed on site.

The results (in log σ_v' versus ε and σ_v' versus M_t format) are shown in Fig. 12 and compared to results of the two CRS tests from this study (from Heimdalsmyra) which had water content closest to that measured at Selva Agdenes. The simulation captures the measured behaviour very well.

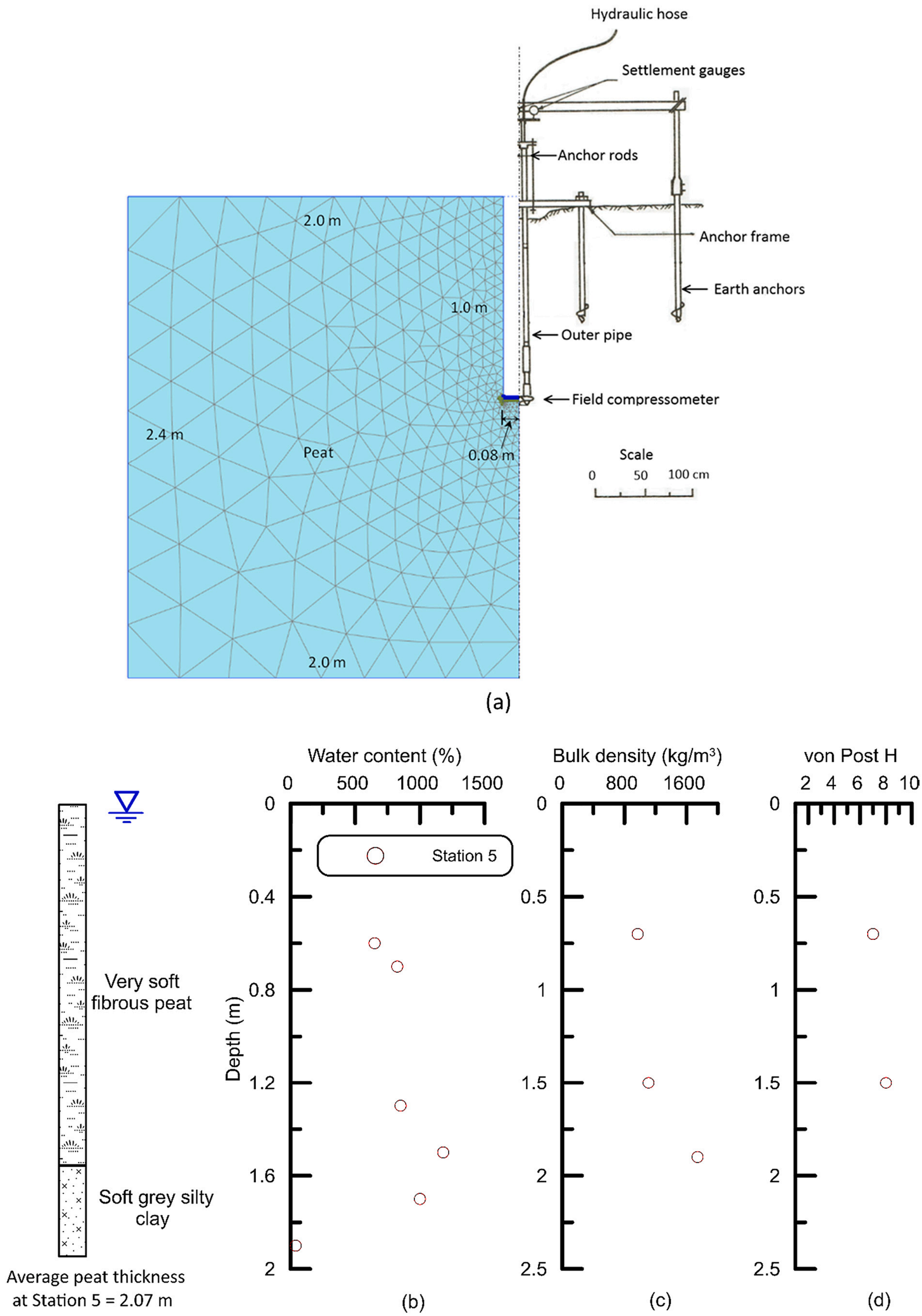

Fig. 11. Heimdlasmyra: (a) field compressometer test set-up showing finite element mesh used in analyses, (b) water content of peat, (c) bulk density and (c) von Post H.

Table 4
Summary of input parameters for SSC model for both sites.

Site	w_i (%)	H (−)	e_0 (−)	γ (kN/m^3)	λ* (−)	κ* (−)	μ* (−)	ν (−)	POP (kPa)	K_0^{NC} (−)	M (−)	φ (deg.)	ψ (deg.)	σ_t (and c) (kPa)	k_{v0} and k_{h0} (m/s)	C_k (−)
Selva-Agdenes	355	5	5.33	10.8	0.24	0.06	0.0144	0.15	5	0.25	2.549	63	30	2.0 (4.0)	2×10^{-7}	1.26
Heimdals-myra	900	7	13.5	10.34	0.24	0.04	0.0144	0	6	0.197	2.785	72	40	3.0 (9.2)	2×10^{-7}	2.90

Notes: w_i: initial water content, H: degree of humification in the von Post scale (average value along the entire peat depth), e_0: initial void ratio, γ: unit weight, λ*: modified compression index in SSC model, κ*: modified recompression index in SSC, μ*: modified creep index v: Poisson's ratio (elastic), POP: Pre-overburden pressure, K_0^{NC}: coefficient of lateral earth pressure in the normally consolidated condition, M: model strength parameter (Critical State Soil Mechanics strength parameter), φ: Friction angle, ψ: dilation angle, σ_t: tension capacity of peat, c: cohesion, k_{v0} and k_{h0}: soil permeability (vertical and horizontal), C_k: rate of change of permeability with increasing stress.

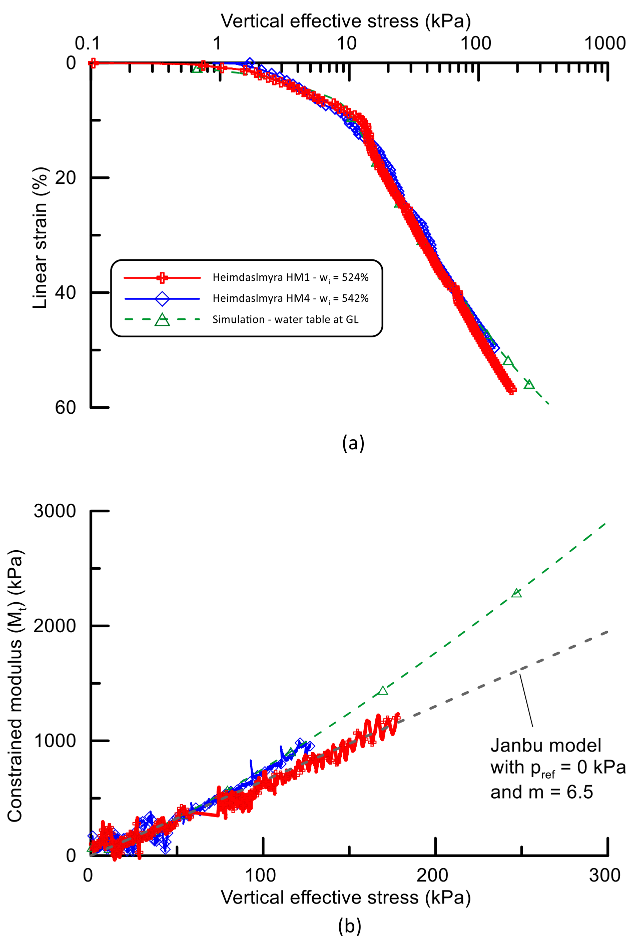

Fig. 12. Simulation of CRS oedometer tests for Selva Agdenes site at 1 m compared to test results with samples of the same water content (a) log σ_v' versus ε and (b) Mt. versus σ_v'.

8.5. Details of full scale simulations

The 2.4 m thick embankment at Selva Agdenes (Fig. 10a) was modelled as being placed gradually over a period of 40 days, before consolidating for the remaining period (a period of 540 days was used in the simulation as this is the period for which data is available). After 540 days the 0.6 m surcharge load was removed in the real situation but was not modelled here as no data was available for this condition. In total 516, 15-noded elements were used in the plane strain analysis with symmetrical conditions assumed around the center line, see Fig. 10a. The silty clay (bottom boundary) was assumed impermeable. Updated Lagrangian analysis was used for both the mesh and the pore pressure (to allow for buoyancy effects). The same three steady state pore pressure profiles, as assumed for the CRS test simulations were assumed.

For the Heimdalsmyra, the 0.16 m diameter field compressometer screw plate was analysed using an axisymmetric model with a circular steel plate of radius 0.09 m and a thickness of 20 mm, see Fig. 11a. The 1 m deep vertical open hole, in which the screw plate was placed, was assumed to be filled with water in the model to mimic that part of the peat disturbed by the installation of the screw plate. Thus it is assumed that the opening created by installation of screw plate is supported by this "mud" pressure. A relatively fine finite element mesh was used with 568, 15-noded elements, in total. Rigid and closed boundaries were assumed at 2.4 m depth and the width of the model was 2 m (Fig. 11a). Rough interface elements were used between the plate and the peat. This interface was extended 30 mm both vertically and horizontally at the edge of the plate. Again the Lagrangian updated mesh option was used.

The stress distribution is dealt with in the finite element method code by consideration of compatibility, equilibrium and the material model. The experiment is challenging to model exactly due to it being a three-dimensional problem with complex boundary conditions. Here it is modelled numerically by approximating the screw plate geometry by an impermeable circular shape. This allowed an axis-symmetric model to be used, as mentioned above.

8.6. Results for analysis of Selva Agdenes embankment

The results of the settlement versus time predictions are compared to the measurements in Fig. 13. Only surface settlement measurements are available for this site. Note that due to construction related conditions on site and the fact that the final measured point (Day No 540) was surveyed some time after the others, the Contractor indicted he placed a little less reliability on this point compared to the others. A reasonable match is obtained between the measured and predicted settlements. Output from the Soft Soil (SS) model in PLAXIS is also shown for comparison. This model is essentially the same as SSC except no creep is allowed for. The SS model predicts less settlement than SSC and this is particularly the case for the long term situation, e.g. the 20 year predictions shown.

It is also interesting to note that the simple empirical method of Carlsten (2000) underpredicts the settlement.

8.7. Results for analysis of Heimdalsmyra field compressometer test

The results of the compressometer load test simulation are compared to the measurements in Fig. 14. In Fig. 14a and b each of the three load steps are shown separately, with linear and log time scales respectively, and in Fig. 14c all three load steps are combined. The final predicted settlement is very close to that measured. However, some differences are noted:

- o The deformation during the first load step is under-predicted by about 1 mm (22%). This is due to the balance in attempting to have as high a G_{max} as possible (by assuming $\nu = 0.0$ as discussed earlier) and as low a κ* as it was reasonable to assume.

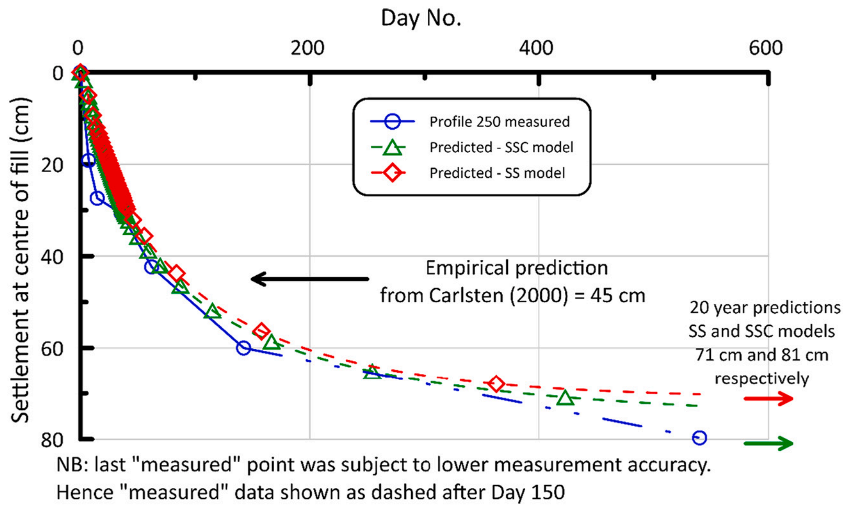

Fig. 13. Measured and predicted settlement for Selva Agdenes embankment.

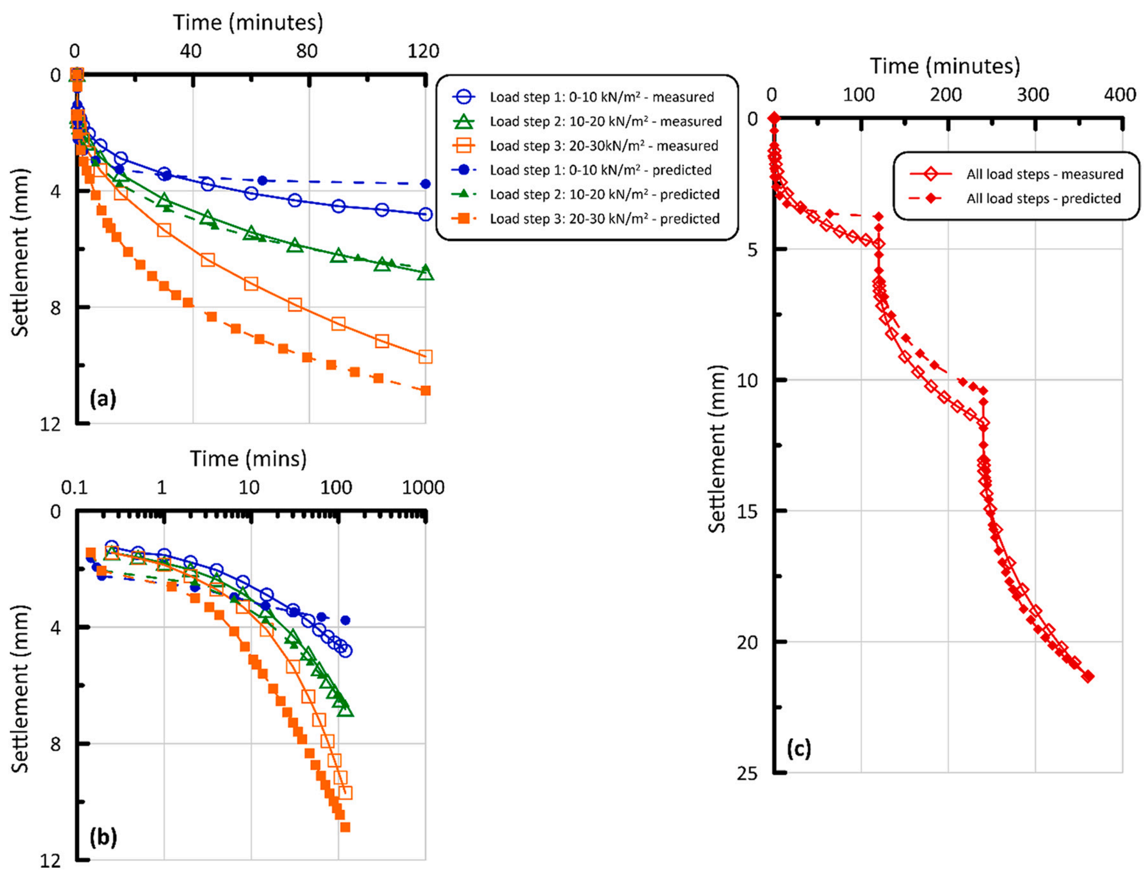

Fig. 14. Measured and predicted settlement for Heimdalsmyra field compressometer test (a) individual load steps linear scale, (b) individual load steps log scale and (c) all load steps combined.

- The prediction for Step 2 is very close to the measured data. This result is encouraging as it involves an application of stress at a level often encountered in practice.
- The third loading step, that takes the stress well above p_{vy}', results in too high settlement predictions. This compensates for the under-prediction in Step 1. This finding is due to the non-linearity in λ^* around p_{vy}' and the requirement that λ^* is matched for total strain past p_{vy}', using a secant value, and not for the incremental strain. It is also noted that the time evolution of settlement is very sensitive to the permeability (and the permeability change) as the drainage path around the edge of the relatively sharp plate assumed in the analysis is likely to have been different from that around the actual more rounded screw plate. This could also partly explain the slightly more delayed response in the field compared to the analysis.

9. Conclusions

Large areas of Norway are underlain by peat. However despite its importance from the point of view of infrastructure construction on peatlands and from a carbon retention perspective its engineering properties are poorly understood. The objective of this paper was to document the characteristics and engineering properties of peat from the Trondheim area to address this research gap. In engineering projects in peat often the resulting settlements are of greatest concern. Therefore here the data obtained from the laboratory tests as well as correlations from the literature are used to make some settlement predictions and these predictions are compared to full scale field measurements. Some conclusions of the study are as follows:

- At the seven Trondheim sites studied the peat is broadly similar, comprising brown fibrous peat with relatively high water content and low degree of decomposition. At one site (Tanemsmyra) the material is unusually thick and contains significant deposits of orange-brown sphagnum peat.
- Despite being visually relatively homogenous the properties of the peat were found to vary significantly over short distances.
- Three different oedometer test types were used. The two test types with standard sized specimens (20 mm high) gave very similar results. The peat in the thicker (50 mm) torvødometer tests consolidated more rapidly perhaps due to some drainage along the sides of the specimen, resulting from the way in which the specimens were prepared. The use of the larger torvødometer sample is motivated by the desire to test a more representative sample of peat compared to the very small samples used in the CRS and IL tests. However the differences in the test results found here warrants further investigation.
- Shear wave velocity measurements both in the field and the laboratory proved very useful for characterising the peat, determining its yield stress and for providing input parameters for the numerical modelling.
- The yield stress of peat is of particular importance in numerical modelling and the determination of the yield stress as reliably as possible was one of the main tasks of this work. There is no single approach appropriate for determining the yield stress of peat. Here four different methods were used, and the average value obtained was shown to be consistent with data and empirical correlations published in the literature.
- The 1D compression parameters (e.g. C_c, C_{sec}, M_0 and M_L obtained from the laboratory tests also compared well with published correlations. Similarly the permeability values were within published limits. However these published limits, for a particular water content or void ratio, can result in a wide variation in the chosen permeability value, perhaps by several orders of magnitude. It is recommended that further work is carried out to study the relationship between index properties and permeability of peat.
- Although the Soft Soil Creep (SSC) model, as employed in this study, is considered to be an advanced model, all of the input parameters can be obtained from the relatively standard tests reported here, empirical correlations found in the literature and from the simulation of element tests (such as an oedometer test).
- Here results of modelling two full scale field cases showed that the SSC model captured reasonably well the vertical settlement versus time behaviour of the peat for the duration of measurement. The peat at the two sites varied between low water content moderately humified peat to high water content highly decomposed peat.
- Particular care needs to be taken in specifying pre-overburden pressure (or yield stress) which is important in deciding the initial strain rate and in selecting stiffness parameters which are appropriate to the stress range and the problem under consideration.
- SSC is a model already used in the practice for modelling settlement of soft soils. The demonstrations presented in this work are considered to contribute towards an effort to encourage practitioners to use this form of modelling for peat and thus bridge the gap between research and practicing engineers.

Data availability

All the laboratory testing and field data generated during this study are available from the corresponding author by request.

Declaration of Competing Interest

We confirm there is no conflict of interest associated with the work presented. It is simply a summary of the results of research project.

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Appendix A. Supplementary data

Supplementary data to this article can be found online at https://doi.org/10.1016/j.enggeo.2022.106799.

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