Engineering Geology 323 (2023) 107226

Contents lists available at ScienceDirect

Engineering Geology

journal homepage: www.elsevier.com/locate/enggeo

Behaviour of 60-year-old trial embankments on peat

Michael Long [a,*], Priscilla Paniagua [b,c], Gustav Grimstad [c], Egil Berg Antoniazzi Sponås [d], Endre Bjertness [e], Stefan Ritter [d]

[a] School of Civil Engineering, University College Dublin (UCD), Ireland
[b] Norwegian Geotechnical Institute (NGI), P.O. Box 5687, Torgarden, 7485 Trondheim, Norway
[c] Norwegian University of Science and Technology (NTNU), 7491 Trondheim, Norway
[d] NGI, P.O. Box 3930, Ullevål Stadion, 0806 Oslo, Norway
[e] COWI, Karvesvingen, 2, 0579 Oslo, Norway

ARTICLE INFO

Keywords:
Peat
Sawdust
Embankment settlement
Numerical modelling
Sustainable solutions

ABSTRACT

The search for low cost solutions for roads on peatlands inspired the construction of four full scale trial embankments at two sites in Norway in the early 1960's. The embankments are still accessible and this work was motivated by present day needs to develop environmentally sustainable solutions to such problems. The embankments were constructed using locally sourced sand and sawdust and have behaved very well with the sawdust settling beneath the water table as designed and remaining in perfect condition some 60 years later. A detailed characterisation of the distinctly different peat at the two sites is presented and it is shown that the peat properties are as expected from local and international experience. Laboratory work on the sawdust showed that it behaves like a highly compressible fibrous peat. Short-term embankment settlement behaviour was controlled by the level of applied load whereas long-term creep settlements were governed by the peat consolidation properties. Pore water pressures dissipated unexpectedly quickly. Modelling using the Soft Soil Creep model in PLAXIS captured both short- and long-term settlements and the pore pressure dissipation well.

1. Introduction

This paper presents some data and analysis of two 60-year-old trial embankments which were constructed in peatlands in Southern Norway. The embankments are still accessible and further investigation was carried out as part of this study. The embankments were built directly on peat using a combination of sand and locally available sawdust. Currently in Norway and other countries there is strong emphasis on finding engineering solutions for infrastructure projects which minimise the environmental effects on the peatlands (Kværner and Snilsberg, 2008). Therefore this study sought to examine the behaviour of these trial embankments with a view to developing low cost and environmentally acceptable solutions for infrastructural projects in peatlands.

The original study in the early 1960's was motivated by the observation that there were many examples of old road and railway systems in peat areas which were upgraded by placing new layers of coarse mineral soils and where the result was an acceleration of settlements and differential settlements (Flaate and Rygg, 1963). A historical solution to the problem was to utilise wood sawdust, chips / shavings and other wood mass as a lightweight fill below the bearing layer. This solution was particularly applicable in this area of Norway where there are many timber based industries. As sawdust has a density some one fifth of that of mineral soils the corresponding settlements will be considerably reduced. Modern solutions, of course, include a variety of techniques and materials such as expanded / extruded polystyrene and foam glass, special lightweight fills such as Leca as well as soil stabilisation techniques such as mass stabilisation, lime-cement columns and others. Some summaries of current practice of treating soft ground in other countries such as the UK and Sweden are given by O'Riordan and Seaman (1993) and Carlsten (1995) respectively.

Similar experience, of these natural lightweight fills, also exists in other countries. The Swedish Road Administration have produced a handbook on using bark and sawdust in road building and give some examples of typical constructions (Vägverket, 1988). Vesterberg et al. (2016) report on work on a low-volume-road project in Sweden where an 80 cm layer of wood chips was used to elevate the road above flood level. Visual inspection five years after construction showed negligible damage from settlement. Hanrahan (1964) reports on the use of

* Corresponding author at: School of Civil Engineering, University College Dublin, Newstead Building, Belfield, Dublin 4, Ireland.
E-mail address: Mike.Long@ucd.ie (M. Long).

https://doi.org/10.1016/j.enggeo.2023.107226
Received 4 January 2023; Received in revised form 4 May 2023; Accepted 19 June 2023
Available online 20 June 2023

lightweight peat bales for the rehabilitation of a road in Ireland. Following reconstruction the road has behaved well (Hanrahan and Rogers, 1981).

In this paper the sites and the trial embankments are described. A detailed characterisation of the geotechnical properties of the peat and the sawdust is given. The performance of the trial embankments from construction up until the present time is outlined. Numerical modelling aiming to back-calculate the field trials and understanding the deformation mechanisms are performed. The overall aim of the paper is to summarise the long-term performance of embankments on peat and to promote the application of these low cost and environmentally sensitive solutions in other similar areas.

2. The sites and the trial embankments

2.1. General

The sites were chosen in conjunction with a roadway project near Kongsvinger and a railway.

project near Oslo. Homogeneous peat deposits were sought and sites were chosen between Sigernessjøen and Motjennet, 10 km south of Kongsvinger, and Slåttemyra, 13 km north of Oslo. The sites are referred to here as Site 1 (Kongsvinger) and Site 2 (Slåttemyra). The work was done in 1963 at Site 1 and in 1964 at Site 2 by the Norwegian Geotechnical Institute (NGI) in cooperation with the Norwegian Public Roads Administration (NPRA) and included two test embankments on each site. The original work is described by Gautschi (1967).

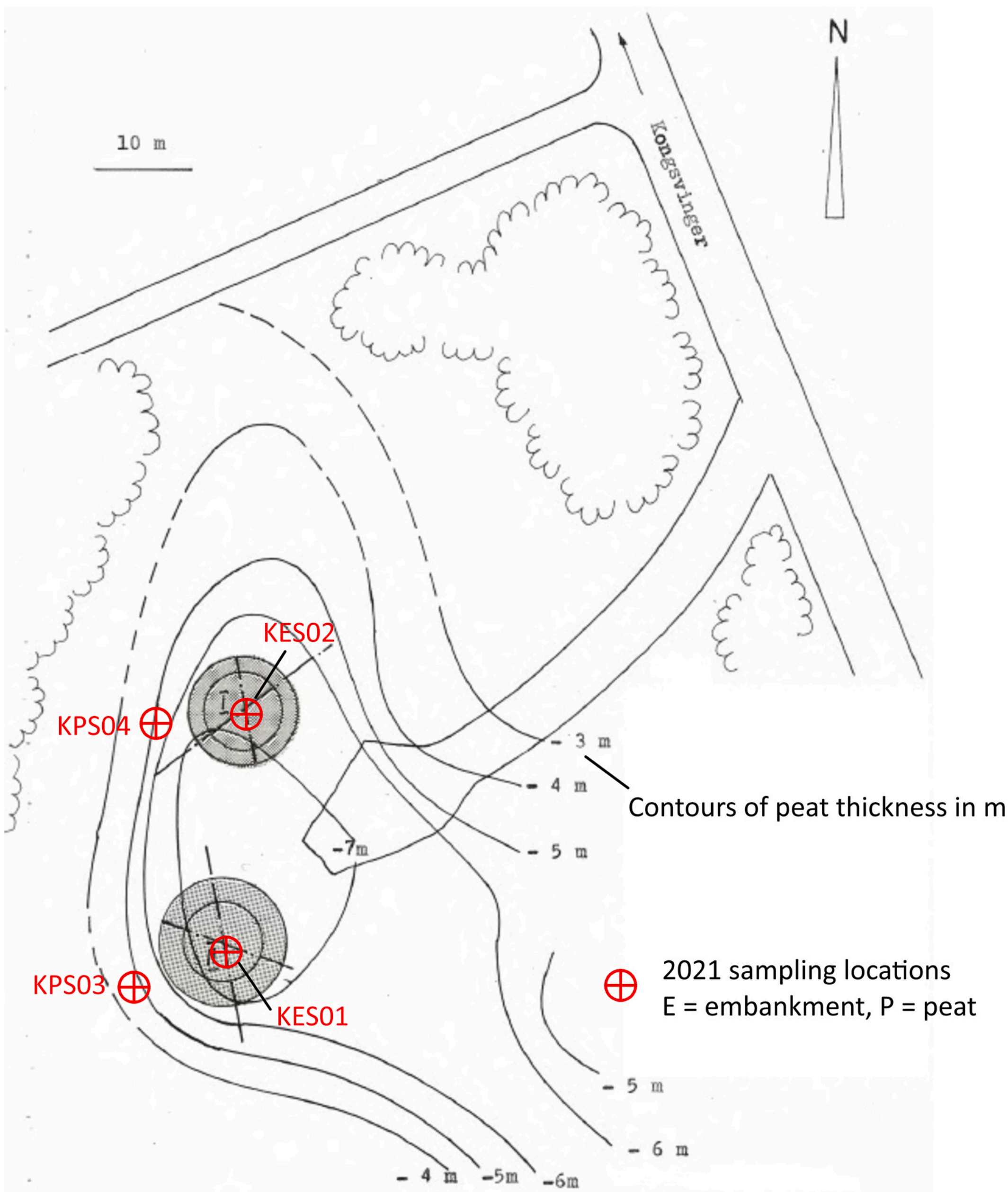

Fig. 1. Layout of test embankments at Site 1 (Kongsvinger) and location of 2021 sampling points. The embankments are located at 60°05′52.4" N and 12°03′57.3″ E. Embankment I to the north and Embankment II to the south.

2.2. Design philosophy

The design philosophy for roads constructed on peatlands using sawdust is described by Flaate and Rygg (1963) and Vägverket (1988). The surface of the peatland is left undisturbed, except for minor clearing of small shrubs and trees. The upper portion of the peat often contains a high percentage of fibres, thus forming a useful working platform. Often a layer of tree logs is placed transverse to the direction of the road and the sawdust is placed on top of the logs. The sawdust has two functions. First it forms a lightweight fill to minimise settlement. Secondly it provides a filter layer to reduce the infiltration of the overlying sand into the peat (perhaps like a modern-day geotextile – which of course were not available at the time of construction). The thickness of the sawdust and sand is chosen so that the resulting settlement will be such to depress the sawdust beneath the water table, therefore maintaining it in a saturated condition long-term and thus minimising the risk of decay. The sand layer forms the bearing surface for traffic.

In the original design the vertical stresses resulting from the conical load of the sawdust were approximated by a uniformly distributed, flexible, circular load. The overlying sand load was also calculated in the same way. For the sand an even distribution through the sawdust layer spread at an angle of 60° was assumed. The stress distribution under the flexible load obtained in this way was determined according to Boussinesq (1885) theory using the tables of Fadum (1948).

2.3. Site 1 - Kongsvinger

The configuration of the two embankments at Site 1 are shown on Fig. 1 and some details of the embankments are summarised on Table 1. The Site 1 embankments are circular and about 2 m high, comprising approximately equal thicknesses of sand over sawdust. They are of the order of 10 m in diameter. The embankment edges were constructed at a slope of 1:1. Peat thickness is approximately 6.5 m. The embankments are clearly visible in the 1967 aerial photographs (https://kart.finn.no/historiske-Eidskog-Kongsvinger-1967). In more recent coverage the embankments are obscured by trees and vegetation.

The embankment construction was done in the months of August and October 1963. Sawdust was delivered from the nearby access road by conveyor belt and distributed by hand. Directly on the peat surface. A few days later the sand was applied in the same way. The aim was to place the sand as quickly to initially create conditions as close to "undrained" as possible. Sand was placed over the course of three hours for Embankment I and five hours for Embankment II. The sawdust and sand had measured in situ density values of 0.6 g/cm^3 and 1.8 g/cm^3 respectively.

2.4. Site 2 - Slåttemyra

The layout of the Site 2 embankments is shown on Fig. 2 and some pertinent details of the embankments are given on Table 1. The Site 2 embankments are of similar construction but of slightly greater diameter than those at Site 1. The peat is a little thinner than that at Site 1. The 2017 aerial photograph (https://kart.finn.no/historiske-Nedre-Romerike-2017) shows the embankments are still clearly visible. The area of the embankments is now designated as a nature reserve.

Construction at Site 2 took place at the beginning of September 1964. The same approach as for Site 1 was used. For Embankment I sand placement took eight hours, while for Embankment II an 80 cm layer was placed on the first day and 95% of the volume had been placed by the end of the second day. Lack of materials required the remaining 5% of the volume to be placed on the third day. The sawdust and sand had measured in situ density values of 0.6 g/cm^3 and 1.7 g/cm^3 respectively.

2.5. Settlement and pore pressure measurement

The settlement of the peat surface both beneath the embankments and away from the toe of the embankments, in addition to that of the top of the sand, was measured by settlement plates. The plates beneath the embankment were connected to the surface by a steel rod. A separate pipe was placed around and concentric to this and was connected to a steel plate placed on top of the sawdust so the settlement of this layer could be measured. Rock outcrops adjacent to the sites were used as benchmarks.

The compression of individual peat layers was measured using anchors installed within the peat mass. These anchors were installed with a steel pipe. Initially the anchor was in a folded position. When the desired testing depth was reached the steel pipe was withdrawn slightly and the anchor was opened out by pulling in the anchor cable. The relative movement of the cables with respect to the surface levels represented the settlement of the intermediate layer.

Some slope indicators, of the type developed at the Swedish Geotechnical Institute (SGI) by Kallstenius and Bergau (1961) were installed at Site 2 to give an indication of lateral movements.

Pore pressures were measured by push in piezometers with a short (7 cm long) ceramic filter tip. The tip was connected to the surface using 6 mm diameter plastic hoses and the water level in the hose was measured using an electrical dip meter.

3. Methodology

3.1. Sampling

Two peat sampling and testing exercise have been carried out. These were the original sampling and testing prior to embankment construction in 1963 / 1964 and a recent sampling exercise in May 2021. The location of the May 2021 sampling points is shown on Figs. 1 and 2.

In the original work a 54 mm diameter steel stationary piston sampler, similar to that described by Andresen and Kolstad (1979), was used. This sampler is still in use today. Gautschi (1967) reports that the samples were "undisturbed" and only those with a water content of approximately 2000% showed any compression due to the sampling process.

In 2021 a peat sampler, similar to that described by Jowsey (1966), was used to obtain a 52 mm diameter "half core" of the peat over the full depth profile. The peat was logged using the extended version of the von Post and Granlund peat classification system (Hobbs, 1986). Small samples were also taken for water content measurements. In addition some undisturbed block samples were taken 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 about 0.3 m

Table 1
Details of trial embankments.

Site	Sawdust thickness (m)	Sand thickness (m)	Peat thickness (m)	Embankment side slopes	Diameter of sawdust (m)	Diameter of sand (m)
Kongsvinger Embankment I (Site 1)	1.0	0.7	6.5	≈ 1:1	10.0	8.3
Kongsvinger Embankment II	1.3	1.2	6.0	≈ 1:1	11.8	9.2
Slåttemyra I (Site 2)	1.2	1.0	6.0	≈ 1:1	11.2	9.0
Slåttemyra II	2.0	2.0	5.0	≈ 1:1	14.0	10.3

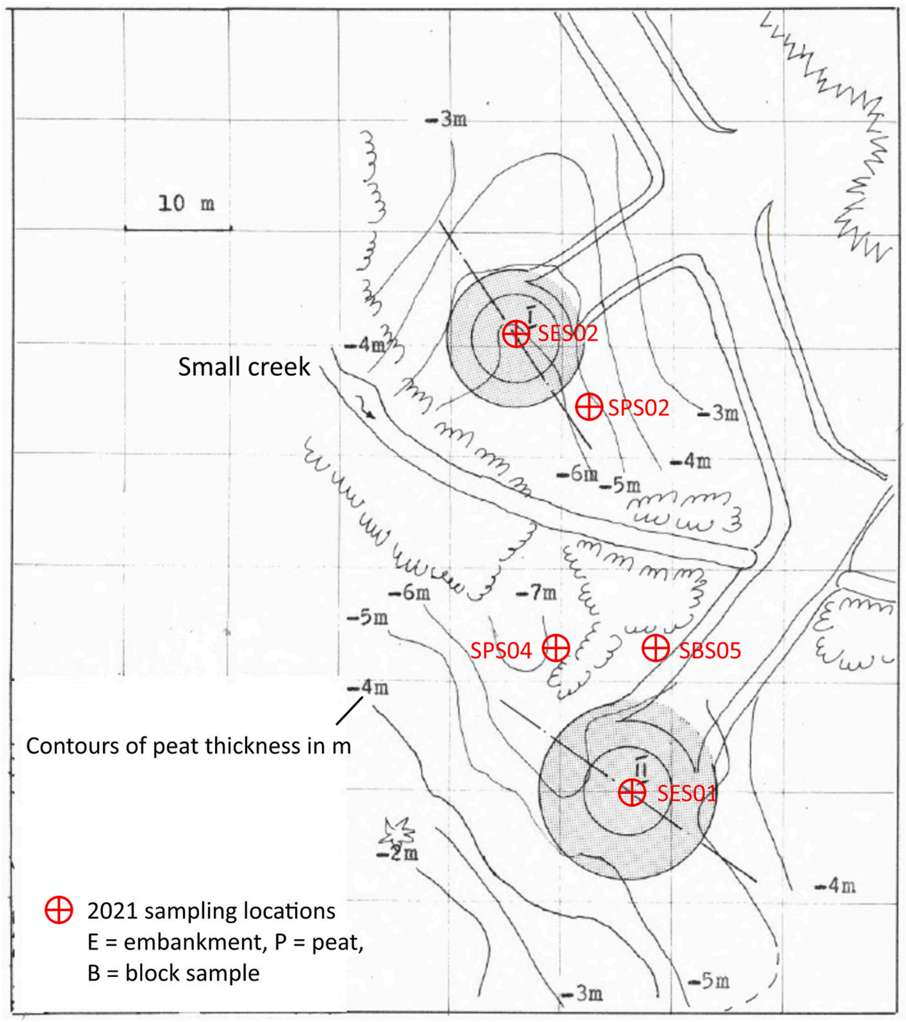

Fig. 2. Layout of test embankments at Site 2 (Slåttemyra) and location of 2021 sampling points. The embankments are located at 60°02′36.8" N and 10°49′58.1″ E. Embankment I to the north and Embankment II to the south.

depth. The sampling procedures followed those recommend by ISSMFE (1981).

As shown on Fig. 1, at Site 1 samples were taken at the location of the embankments (KES01 and KES02) and from natural ground adjacent to the embankments (KPS03 and KPS04). A hand carved block sample was also taken at location KPS03. At location KES01 it was not possible to penetrate through the sand and sawdust. Peat was recovered at all other locations but the base of the peat was not proven due to sampler penetration resistance.

Similarly at Site 2 (Fig. 2) samples were taken from beneath the embankment (SES01, SES02) and from the undisturbed bog surface adjacent to the embankments (SPS03 and SPS04). At SES01 and SES02 no peat was recovered. Again resistance to penetration prevented the base of the peat being identified at all locations. A block sample was taken at location SBS05.

3.2. Index testing

Index testing for the peat and sawdust materials was carried out using standard Norwegian practice (NPRA, 2014), with some pertinent features of the tests as follows:

- o Water content: oven dried at 50 °C for at least 48 h,
- o Loss on ignition (LOI): 950 °C for four to five hours,
- o Particle density: pycnometer method,
- o Minimum and maximum density used methods outlined by Lunne et al. (2019).

3.3. Laboratory oedometer testing

Continuous rate of strain (CRS) oedometer tests were carried out on the block samples of peat excavated from both sites in 2021 at NTNU. These tests were undertaken on 50 mm diameter by 20 mm high samples using the standard NGI procedures outlined by Sandbaekken et al. (1986). Long et al. (2022a, 2022b) demonstrated that the chosen strain rate in CRS tests has little effect on the stress – strain and stiffness behaviour of the peat. Therefore a commonly used and practical strain rate of 3% strain / hour was adopted.

Both incrementally loaded (IL) and CRS oedometer tests were carried out on sawdust samples. Full detail are presented by Bjertness and Sponås (2021). Again standard NGI procedures were adopted. Samples were built in by tamping sawdust in layers.

4. Ground conditions and geotechnical parameters of peat

4.1. Index properties – Site 1

The peat at Site 1 can be divided into three layers as shown on Fig. 3. The upper peat layer can be described as a very soft brown fibrous sphagnum peat. It has an average von Post H and water content of about

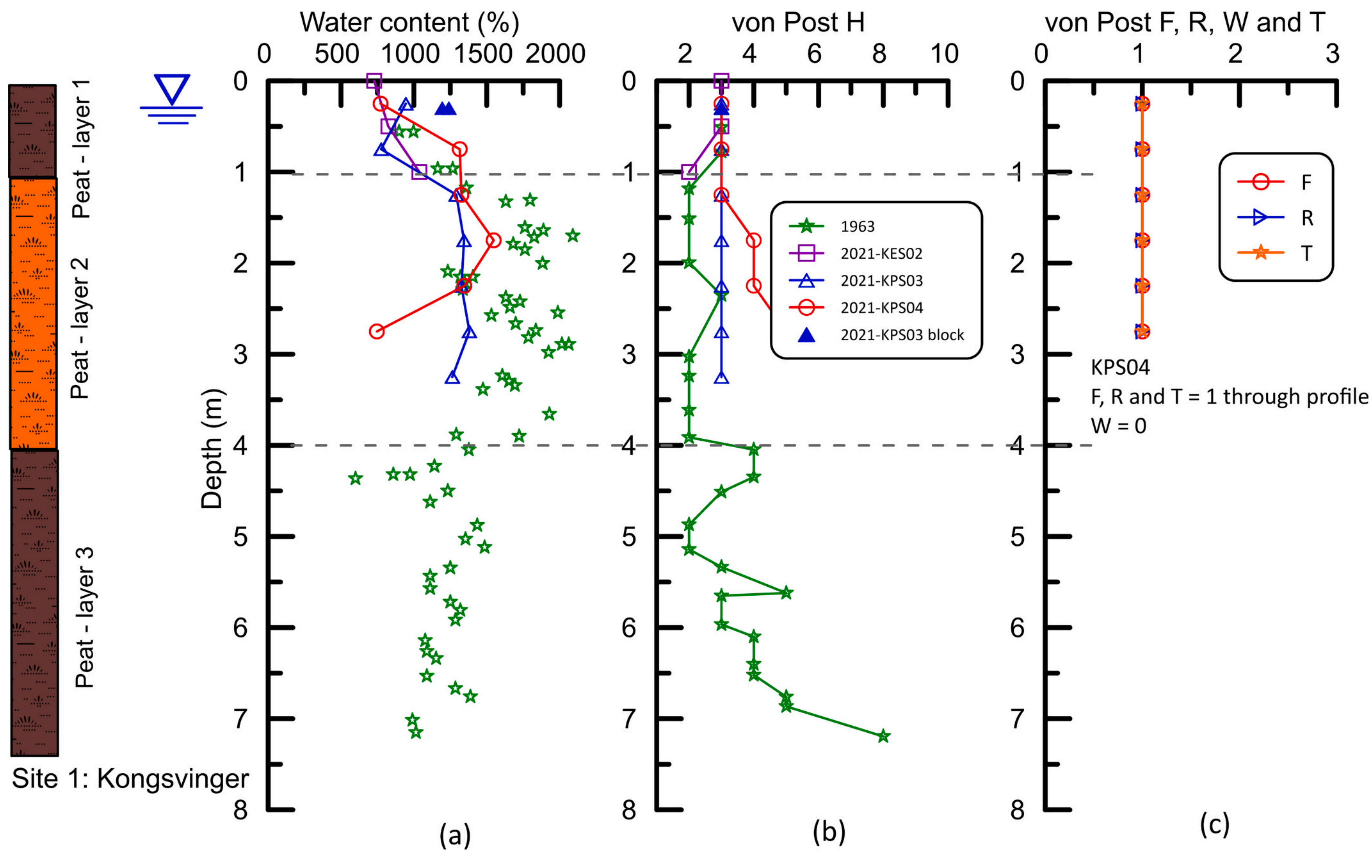

Fig. 3. Peat profile at Site 1 along with (a) water content, (b) von Post degree of decomposition (H) and (c) von Post fine fibre (F), coarse fibre (R), wood fraction (W) and tensile strength (T) indices.

3 and 1000% respectively. The middle peat layer has a dark brown / black colour, a similar average H to the upper layer of about 3 but has a much higher average water content of some 1675%. In the lowest peat layer the water content drops to an average of about 1165% and the peat becomes more decomposed with depth. The peat has average bulk density of 1.025 Mg/m^3, i.e. just slightly greater than that of water.

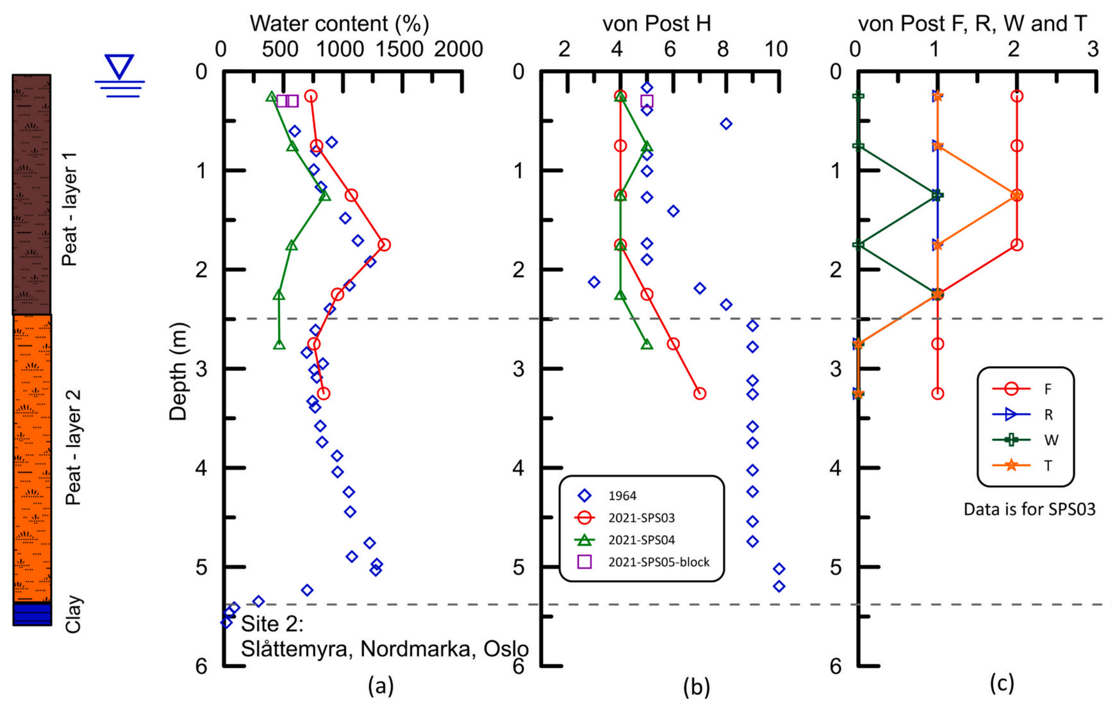

Fig. 4. Peat profile at Site 2 along with (a) water content, (b) von Post degree of decomposition (H) and (c) von Post fine fibre (F), coarse fibre (R), wood fraction (W) and tensile strength (T) indices.

Perhaps not surprisingly the 2021 data show lower average water contents than that from 1963 most likely due to drainage induced by loading of the embankments. A full von Post log for the 2021 samples is shown on Fig. 3c. This comprises H along with an estimate of the fine fibre content (F), the coarse fibre content (R), the wood content (W) as well as the tensile strength (T). The F, R and T indices were estimated to be 1 (range: 0–3) throughout the profile. There was no discernible wood fraction.

4.2. Index properties – Site 2

The peat at the Site 2 site is distinctly different from that at Site 1 (Fig. 4). Overall the peat has higher degree of decomposition and lower water content than that at Site 1. Visually the peat can be described as brown becoming dark brown to black fibrous peat. There is no obvious sphagnum content. There are two distinct peat layers. The upper layer has average H, water content and organic content of about 5, 920% and 85% respectively. The lower layer has higher degree of decomposition, a water content which increases with depth from about 750% to 1275% and an average organic content of 63%. The 1964 investigation suggests that the peat has a higher degree of decomposition than in the 2021 investigation. This difference is very likely to be due to the subjective nature of the von Post classification technique. The important finding is that the peat at Site 2 is different and more decomposed than that at Site 1.

Again the 2021 data shows lower water content than that from 1964, perhaps due to drainage effects. The full von Post log on Fig. 4c shows that the peat at Site 2 has more appreciable fine fibre and wood content than that at Site 1 and shows greater tensile strength.

4.3. 1D consolidation properties – both sites

Some CRS test results for peat from the two sites peat are shown on Fig. 5. Results are presented in classic log effective axial stress (log σ_v') versus axial strain (ε) form and as σ_v' versus constrained modulus (M_t), where:

$$M_t = \frac{\Delta \sigma_v'}{\Delta \varepsilon} \tag{1}$$

The water content (w) and von Post H are also shown on the figure. Two tests were caried out for each of the peats and the results show good repeatability.

Some interpreted engineering parameters such as the modulus at in situ stress (M_0), the yield stress (p_{vy}'), the compression index (C_c) and the modulus number m (i.e. the slope of the M_t versus σ_v' line beyond the yield stress) are also indicated on the plot. The higher water content and less decomposed Site 1 peat shows lower yield stress and greater compressibility than that from Site 2. The derived engineering parameters are consistent with the trends for peat in the Trondheim area of Norway (Long et al., 2022a, 2022b).

4.4. Permeability - both sites

A plot of permeability value (k_v) as derived from oedometer tests from the Site 1 site (Gautschi, 1967) are shown on Fig. 6. The values were derived from oedometer tests results. Permeability values are very variable but there is some trend of decreasing permeability with increasing void ratio (e) as would be expected. There is no clear relationship between permeability and degree of decomposition, as expressed by the von Post H. The data also falls within the upper and lower bound limits suggested by Mesri and Ajlouni (2007).

5. Geotechnical properties of sawdust

Gautschi (1967) describes the lightweight material used at Site 1 to be "sawdust" and that at Site 2 to comprise "small shavings". Locally

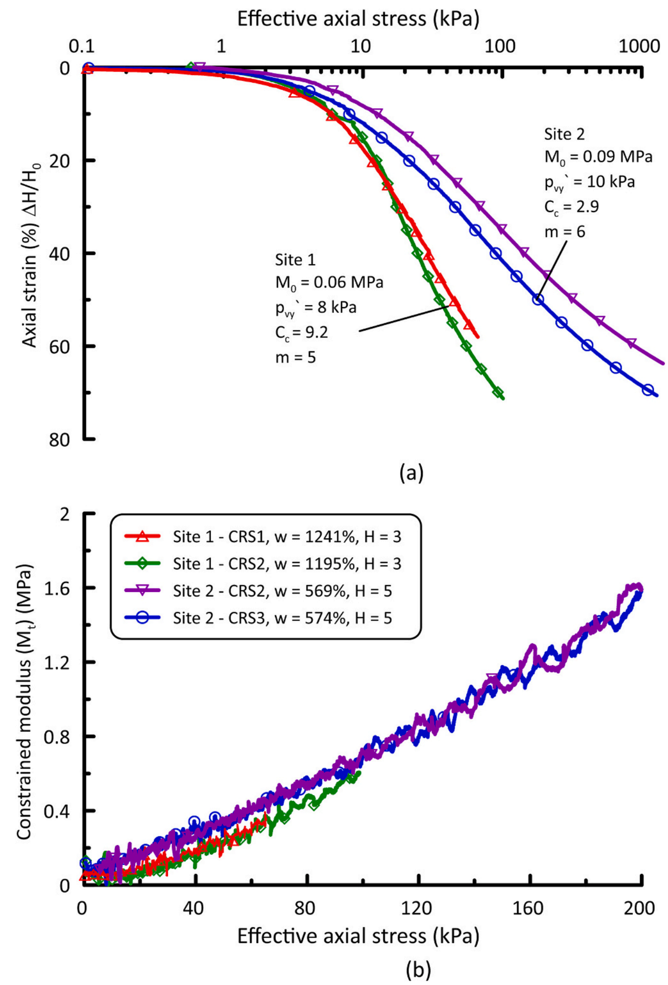

Fig. 5. Constant rate of strain (CRS) oedometer tests results for both sites (a) log σ_v' versus ε and (b) σ_v' versus M_t.

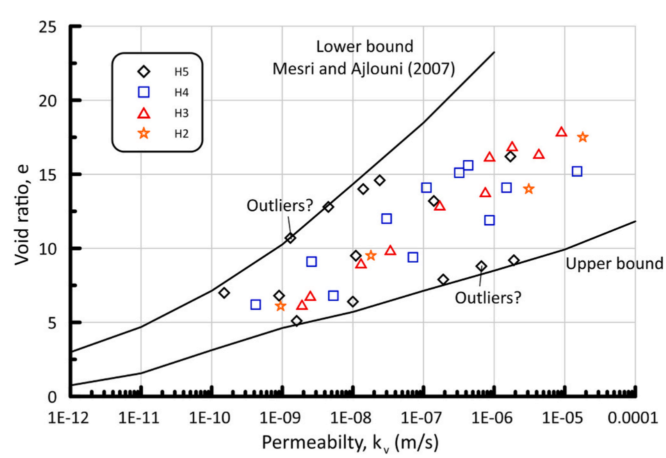

Fig. 6. Permeability of peat at Site 1 showing degree of decomposition

available product was used in both cases. Although some material was recovered in-situ during the 2021 investigation it was necessary to source additional similar material to permit supplementary laboratory testing. A full account of the properties and testing of the sawdust materials is given by Bjertness and Sponås (2021) and is summarised here. Of the four materials studied the sawdust used in Site 1 was found to have similar gradation to a product known as small wood chips (WcS), originating from spruce trees, from Borregaard biorefinery in Sarpsborg, southern Norway. The small shavings used at Site 2 were also similar to the WcS and to a second spruce derived sawdust material (SdMn) obtained from Moelven Soknabruket AS located at Sokna in Ringerike, southern Norway.

The sawdust properties are detailed on Table 2 and are briefly summarised here.

5.1. Density

The results are influenced by the method of preparing the sample. The samples which were prepared dry have much lower density than those prepared in a saturated state. In some cases, for the WcS sawdust, the maximum dry density determined using the vibratory method (Lunne et al., 2019) is less than the actual measured dry density in oedometer samples. This is due to the vibratory method, which involves oven drying of the material prior to testing, being originally designed for sand.

The in situ measured density values are consistent with the material being placed in a moist / wet condition.

Inspection of the sawdust particles recovered from beneath the embankments suggests that they have become stiffer from a hardening process due to ageing.

Table 2
Summary of properties of sawdust.

Property	Unit	Site 1 in-situ	Site 2 in-situ	WcS	SdMn
Water content	%	150–200	150–200		
Particle density	g/cm^3	1.42	–	1.5	1.5
Loss on ignition	%	86.9	72.7	99.5	99.7
Particle size distribution					
D_{10}	mm	–	–	0.63	0.44
D_{25}	mm	–	–	1.30	0.80
D_{50}	mm	–	–	1.60	1.50
D_{60}	mm	–	–	1.83	1.79
D_{75}	mm	–	–	2.34	2.50
Coefficient of uniformity, C_u		–	–	2.9	4.1
Density (prepared dry)	g/cm^3	–	–	0.19–0.23	0.21–0.27
Density (prepared wet)	g/cm^3	0.6	0.6	–	0.47–0.76
Minimum dry density (method dependant)	g/cm^3	–	–	0.11	0.12–0.19
Maximum dry density (method dependant)	g/cm^3	–	–	0.19	0.22–0.32
Unloading-reloading modulus, M_{ur}	kPa	–	–	3300–4000	2000–4000
Modulus number, m		–	–	5–8	2–7
C_c	–			2.7	2.4
Coefficient of secondary compression, C_α	–	–	–	0.075	0.05–0.09
C_α /C_c				0.028	0.025
Permeability, k	m/s			In the order of 10^{-6}	In the order of 10^{-6}
Effective friction angle, ϕ’	Deg.	–	–	–	Very high > 50
Effective cohesion, c’	kPa	–	–	–	0

5.2. 1D compression parameters

The results of IL tests are presented here. CRS tests were also caried out (Bjertness and Sponås, 2021) and showed similar results. The IL tests are more complete as they also included measurements of permeability using the falling head method. Similar to the peat CRS tests results are presented in the form of σ_v’ versus ε and σ_v’ versus M_t on Fig. 7a and b respectively. The water content (w) and dry density (ρ_d) of the specimens is also noted on the figure.

Two of the specimens were built in after drying and two had water contents as obtained from the suppliers. All specimens showed behaviour similar to that of a sand. The drier samples showed a slightly stiffer response. The stiffness in the unload – reload loop was much higher than that from initial loading.

5.3. Creep

Creep parameter (C_α) is defined as the change in void ratio divided by the change in log time:

$$C_\alpha = \frac{\Delta e}{\Delta logt} \quad (2)$$

This parameter was on average 0.06 for the stress range encountered in the field at the two sites. Many engineers refer to 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. Here $C_\alpha/C_c = 0.025 \pm 0.01$.

5.4. Permeability

Permeability values were measured using the falling head method in the incrementally loaded oedometer tests, see Fig. 7b. For the WcS material the values varied between 1×10^{-6} m/s at low stress to about 3×10^{-7} m/s at 80 kPa. The relationship between the change in permeability to the change in strain is characterised by the parameter β, where:

$$\beta = \frac{\Delta logk}{\Delta \varepsilon} \quad (3)$$

For the WcS material, β was equal to 5.6. For the SdMn values the equivalent range was 2.3×10^{-6} m/s and 5×10^{-7} m/s.

5.5. Shear strength

Some isotropically consolidated undrained triaxial tests on saturated SdMn sawdust were carried out by compacting the material by hand into the device. The resulting water content and density was of the order of 300% and 0.76 g/cm^3 respectively. The samples were 5.8 cm in diameter and 11.4 cm high. The stress versus strain plot shows that the material behaves in a dilatory manner with shear stress increasing continuously with strain. Pore pressures develop very rapidly, resulting in the stress path reaching the tension cut-off line at a shear stress of some 10 kPa. The resulting friction angle was very high and the material exhibited little effective cohesion. Overall the behaviour of the sawdust is very similar to that of fibrous peat (Lee et al., 2015; O'Kelly, 2017).

6. Embankment performance

6.1. Settlement and pore water pressure at embankment centre

Measured values for the excess pore water pressure 2 m below the peat surface and the settlement at the top of the peat at the centre of the embankments, as reported by Gautschi (1967), are shown on Fig. 8a and b for Site 1 and 8c and 8d for Site 2.

The settlements were higher for Site 2 than for Site 1. This is consistent with the higher embankments at Site 2 (2.2 m and 4 m compared to 1.7 m and 2.5 m at Site 1). Site 1 peat has higher water

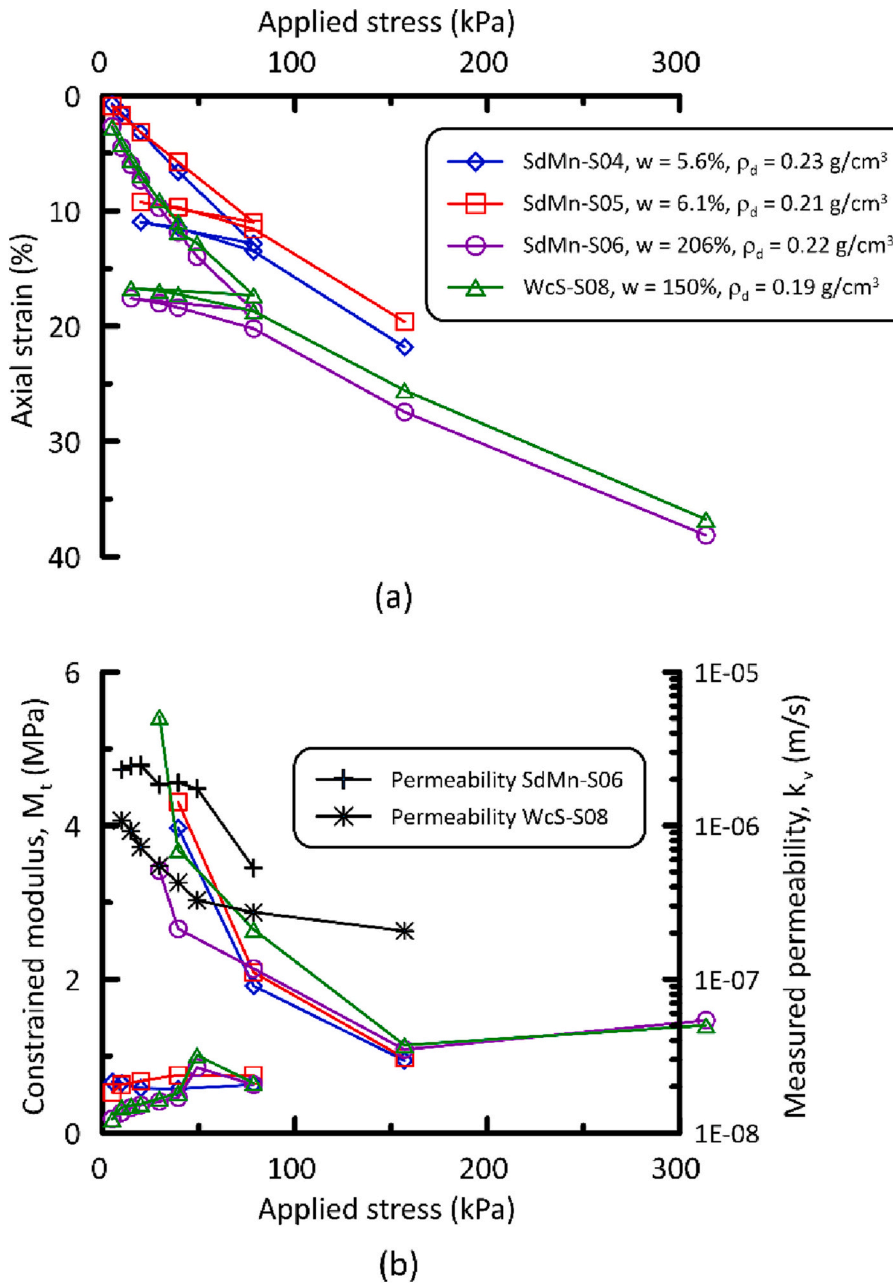

Fig. 7. Incrementally loaded oedometer tests on sawdust (a) σ_v' versus ε and (b) σ_v' versus M_t and k_v.

content but is less decomposed than that at Site 2. In both cases the settlements were higher for Embankment II, which were the highest embankments and ranged between 135 cm at Site 1 and 187 cm at Site 2. Ongoing creep is evident, past the monitoring period, with perhaps that at Site 1 being greater than that at Site 2. Maximum settlements are summarised on Table 2.

A significant feature of the embankment performance is the relatively rapid dissipation of excess pore water pressure, especially at Site 2, where the excess pore pressure had essentially dissipated after some 20 days. This finding is perhaps inconsistent with the data shown on Fig. 6 which suggests that a lower water content peat should have lower permeability. The influence of the chosen permeability value is explored later in the numerical modelling section.

6.2. Compression of peat layers

The compression of individual peat layers for both sites are shown on Fig. 9a and b for Sites 1 and 2 respectively. For Site 1 the compression of all three layers is approximately equal. The upper two layers are about 1.5 m thick and the lowest layer is 3 m thick. These findings are consistent with the upper two layers having approximately equal water content (1685% on average, see Fig. 3), with the lowest layer having a lower average water content of 1165%.

For Site 2 the upper layer has the lowest water content (see Fig. 4) and this layer correspondingly shows the lowest compression. The layer between 1 m to 2 m has the highest average water content and shows the highest relative compression. The settlement of the other layers is also consistent with their thickness and water content.

6.3. Settlement profile across embankments

The transverse settlement profile for Site 1 Embankment I is shown on Fig. 10. This particular embankment is chosen for illustration here as long-term settlement at the centre of this embankment is known from

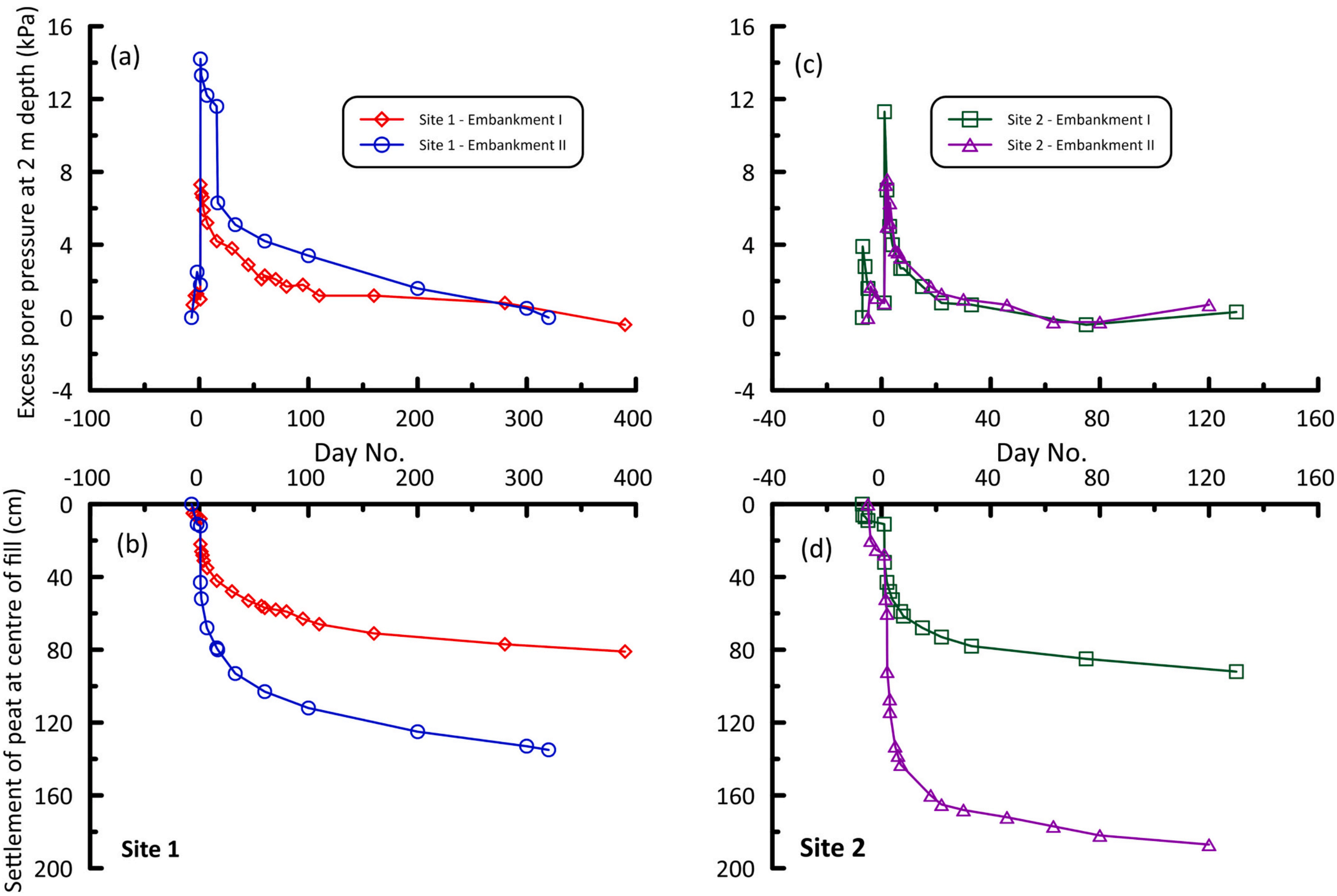

Fig. 8. Measured excess pore pressure at 2 m depth and settlement of peat surface at centre of embankment at Site 1 (a and b) and at Site 2 (c and d). Note x-axis scales changed for clarity of display of data.

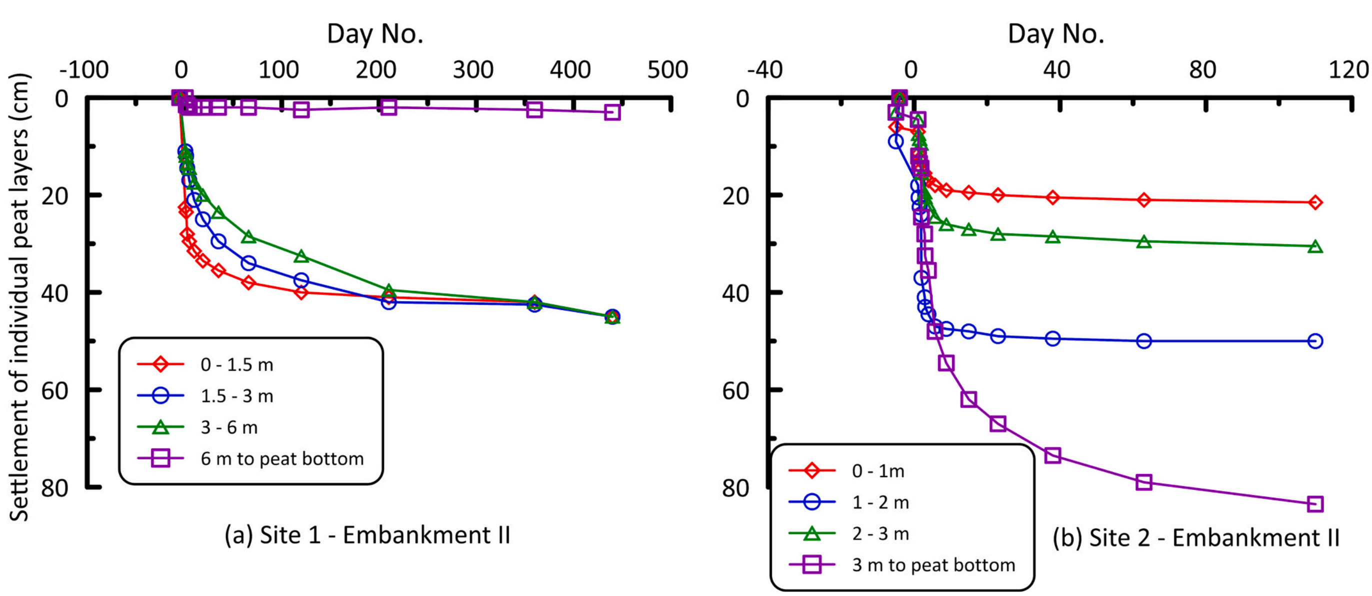

Fig. 9. Compression of individual peat layers (a) Site 1 Embankment II and (b) Site 2 Embankment II. Note x-axis scales changed for clarity of display of data.

the May 2021 investigation. For the other three embankments it was not possible to drill through the sawdust and sand layer to identify the top of the peat. The original as built dimensions are based on (Gautschi, 1967) and the current state was obtained from www.hoydedata.no, with layering as determined from the field investigations in May 2021.

The maximum settlement of the original peat surface increased from approximately 81 cm at the end of the monitoring period to about 120 cm 58 years later. The 1.0 m thick sawdust layer was not completely submerged at the end of the monitoring period but subsequently settled beneath the water table. The sawdust thickness reduced from 1.0 m to

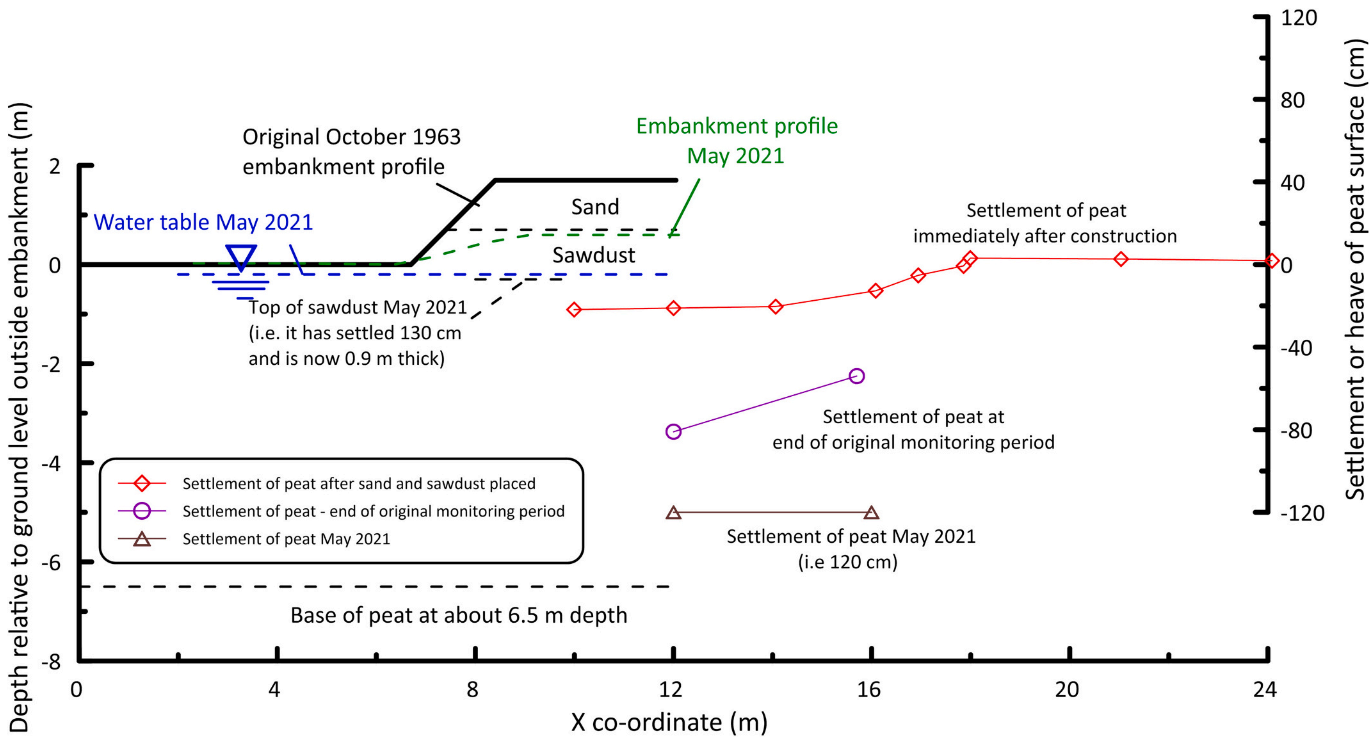

Fig. 10. Transverse profile and settlement across Embankment I at Site 1.

0.9 m during the lifetime of the embankment.

Similar profiles for the other three embankment are given by Bjertness and Sponås (2021). The performance can be summarised on Table 3. The sawdust was fully submerged at the time of the May 2021 investigation for three of the four embankments. For Site 1 Embankment II the sawdust was fully submerged at the end of the monitoring period in 1964.

6.4. Heave of peat surface

Heave movements of the peat surface away from the toe of the embankments were also observed. For Site 1 the maximum recorded values were 3.3 cm for Embankment I at 2.5 m from the toe of the embankment and 5 cm for Embankment II 3.5 m from the toe. For Site 2 the corresponding values were 1 cm and 7.5 cm both recorded 4 m for the embankment toe.

6.5. Present condition of the embankments

Some images showing the embankments over the years since construction are shown on Fig. 11. All four embankments are clearly visible in the aerial photographs from 1967 and 1976 (Fig. 11a and d). Vegetation quickly developed on Site 1, whereas Site 2 remained in a similar condition to when the embankments were constructed, as can be seen on Fig. 11b and e respectively. When the sites were visited in May 2021 they both appear to be in a quite similar condition to that described by Gautschi (1967). Besides a new road section close to the embankments at Site 1 and generally denser vegetation (Fig. 11c and f respectively), there is no sign of activities that have affected the settlement of the embankments. The hydrological conditions are believed to be more or less unchanged since construction with the exception of possible minor erosion of the creek at Site 2.

Growth of trees has occurred on all the embankments, primarily of spruce, but also pines, birch, and shrubs are observed. With the exception of Site 2 Embankment II, the perimeter (slope) of the embankments were so densely grown with trees that it was difficult to see the embankments and get access. The embankment surfaces are generally covered with dead plant material, thin moss and grass.

A remarkable feature of conditions at both sites is that the 57 to 58 year-old sawdust was well preserved (Fig. 12) and appeared intact in texture and colour. Unlike fresh sawdust the sampled sawdust was odourless. As has been discussed above, the sawdust layer had settled below the ground water table. The sawdust in Embankment I at Site 1 was discoloured at the top of the layer and appeared paler with depth. At the peat boundary it was hard to distinguish peat from sawdust because they both were pale in colour and almost identical. Peat was thus identified only when squeezing the material from the auger. Whereas the sawdust was firm, the peat compressed easily and released clear water when squeezing in the hand.

The sand in Site 1 embankments contained more coarse grains (fine gravel) than that in Site 2. Roots of significant thickness were observed up to 1 m depth below the surface.

Table 3
Summary of maximum settlement and long-term observations.

Embankment	Sand thickness (m)	Sawdust thickness (m)	Max. monitored settlement (cm)	Water table May 2021 (m below original ground level)	Top of sawdust May 2021 (m below original ground level)	Top of peat May 2021 (m below original ground level)
Site 1- I	0.7	1.0	81	0.2	0.3	1.2
Site 1 - II	1.2	1.3	135	0.2	0.7	–
Site 2- I	1.0	1.2	92	0.5	0.95	–
Site 2- II	2.0	2.0	187	0.1	−0.2	–

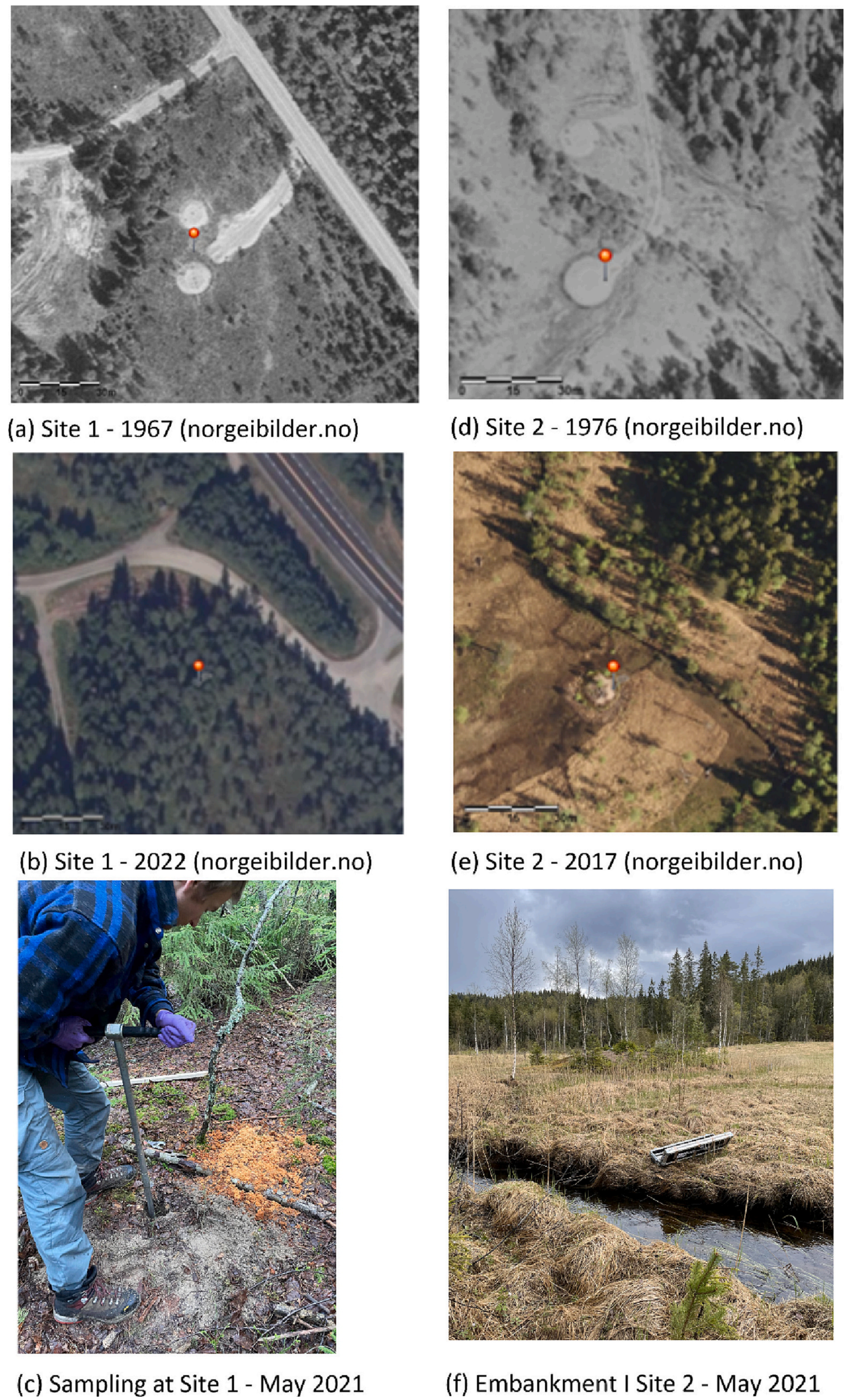

Fig. 11. Images showing condition of embankments since construction.

Fig. 12. Photograph of sawdust and sand in auger extracted from Site 1 Embankment II in May 2021

7. Modelling of behaviour

7.1. Choice of constitutive model and general modelling parameters

Here the Soft Soil Creep (SSC) model in the finite element code PLAXIS (2020) is used to predict the long-term settlements of Site 1 Embankment I. This embankment was selected as the long-term settlement of the peat surface is known. The model was chosen as it was important to incorporate creep into the calculations given the settlements occurred over a period of some 60 years. SSC has been previously used successfully to estimate settlements in peat in the Netherlands (Den Haan and Feddema, 2013), Sweden (Long et al., 2022a, 2022b) and Norway (Long et al., 2022a, 2022b). The reader is referred to these papers and the PLAXIS manual for further details.

The geometry of the embankment is shown on Fig. 10. An axisymmetric updated Lagrangian analysis was used for both the mesh and the pore water pressure (to allow for buoyancy effects). This form of analysis was essential due to the large deformations involved. The bottom boundary is fully fixed mechanically and left and right boundaries are mechanically fixed in the horizontal direction and free vertically. The model had 922 15-noded soil elements. Groundwater pressures were initially assumed to be hydrostatic from ground level. The symmetry line and the bottom boundary were assumed closed for flow but the top and the sides were kept open.

In the simulation the embankment is assumed to have been constructed in 1 day using a coupled (flow / deformation) analysis, then left to consolidate up to present (time of last measurement).

7.2. Assumptions made when choosing peat properties

A summary of the parameters assumed for the three layers of peat (Fig. 3), all of which were modelled using SSC is given on Table 4. Some specific assumptions made are as follows:

- o The model strength parameter, M, was set to 3.0 using a K_0^{NC} (the coefficient of lateral earth pressure in the normally consolidated condition) of about 0.165.
- o A tensile strength of 2 kPa was assumed (Dykes, 2008).
- o The Coulomb friction angle (φ), dilatancy angle (ψ) and cohesion (c) were set to match the tension cut-off. This combination combines the "wet" side of a rate dependent modified Cam Clay model with a tension cut-off criterion.
- o In SCC use is made of the modified compression index (λ*):

$$\lambda^* = \frac{C_c}{2.3(1+e)} \tag{4}$$

According to the odeometer tests on Site 1 peat (Fig. 5) λ* is in the range 0.2 to 0.25, with 0.25 being relevant to the stress range in this problem.

- o Similarly the modified recompression index (κ*) is given by:

$$\kappa^* \approx \frac{2C_s}{2.3(1+e)} \tag{5}$$

where C_s is the swelling index. κ* was set to λ*/10 (Mesri and Ajlouni, 2007).

- o As the objective here is to predict long term settlements the creep parameters need to be chosen with care. In SCC use is made of the modified creep index (μ*):

$$\mu^* = \frac{C_\alpha}{2.3(1+e)} \tag{6}$$

Here it was assumed μ* = 0.07λ*. This corresponds to the upper estimate from Mesri and Godlewski (1977) and Mesri and Ajlouni (2007) for peat with low von Post H. Waterman and Broere (2005) recommend that the user should carefully consider the creep ratio: (λ* − κ*) / μ*, which ideally should fall within the range of 10 to 20 for peat. Here the value used is 12.9.

- o An apparent pre-consolidation effect in the peat is introduced through a pre-overburden pressure (POP) of 8 kPa, consistent with the oedometer tests (Fig. 5).
- o As can be seen from Fig. 6, the value of the permeability value is highly uncertain. Therefore this value was varied in the analyses to check the sensitivity of the output to the chosen input. Values were

Table 4
Summary of input parameters for Soft Soil Creep (SSC) model for peat.

Material	w_i (%)	H (−)	e_0 (−)	γ_{sat} (kN/m^3)	λ* (−)	κ* (−)	μ* (−)	ν_{ur} (−)	POP (kPa)	K_0^{NC} (−)	M (−)	φ (deg.)	ψ (deg.)	σ_t (kPa)	k_{v0} and k_{h0} (m/s)	C_k (−)
Peat layer 1	1080	3	16	10.3	0.25	0.025	0.0175	0.1	8	0.165	3.0	≈90	≈90	2.0	2×10^{-6}	3.5
Peat layer 2	1685	2	25	10.2	0.25	0.025	0.0175	0.1	8	0.165	3.0	≈90	≈90	2.0	1×10^{-5}	6.0
Peat layer 3	1165	4	17.5	10.3	0.25	0.025	0.0175	0.1	8	0.165	3.0	≈90	≈90	2.0	5×10^{-6}	3.5

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_{sat}: saturated unit weight, λ*: modified compression index in SSC model, κ*: modified recompression index in SSC, μ*: modified creep index, ν_{ur}: 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 = $\sigma_t$cotφ, k_{v0} and k_{h0}: soil permeability (vertical and horizontal), C_k: rate of change of permeability with increasing stress.

originally chosen from Fig. 6 using the corresponding void ratio values for each peat layer (Table 4). In subsequent analyses the values were divided by 10 and 100 respectively.

7.3. Parameters for sawdust and sand

The sawdust and sand were modelled using the Mohr-Coulomb (MC) constitutive model. The behaviour of sawdust is undoubtedly much more complex than that which the MC model can capture. However, for the purpose of a settlement analysis, with a focus on the peat, it is believed to give sufficient accuracy. The unit weight of the sawdust is particularly difficult to model in completely realistic way, as the degree of saturation in the sawdust above the original ground water level (GWL) is expected to change both due to capillarity and precipitation. For the analysis a fixed value, close to the original ground level, only is used. This might underestimate the total weight of the embankment until the sawdust becomes fully submerged.

A summary of the input parameters for the sand and sawdust is given on Table 5.

7.4. Results of the analysis

Except for the permeability, no attempts have been made to alter the input parameters to match the measured settlements and pore water pressures. The chosen values were determined from the laboratory tests and empirical correlations as discussed above and on Tables 4 and 5. The actual versus measured pore pressures and settlements are shown on Fig. 13.

The calculated settlement of the peat surface versus time matches very well with the measurements both during the original monitoring period and in the long-term. The model results where the original permeability values were dived by 10 match the measurements particularly well. The analysis suggests that the settlement at the top of the peat is divided roughly evenly between the three peat layers from 0 to 1.5 m, 1.5 m to 3 m and 3 m to 6 m. This is consistent with the finding for Site 1 Embankment II as shown on Fig. 9. (Unfortunately the equivalent data is not available for Embankment I.)

For the originally chosen permeability values, the simulated excess pore pressure dissipates faster than the in-situ measurements indicate. However when the input values are factored down by 10 the match between measurements and predictions is reasonably good.

It should be noted that no maintenance was carried out over the years since the embankments were constructed. The influence of the vegetation which grew on the sites has not been included in the analysis. For the case of an active full-scale structure regular maintenance would, of course, have been carried out.

8. Conclusions

The main purpose of this paper is to promote the use of low-cost sustainable infrastructure in peatlands by documenting the behaviour of four 60-year full-scale trial embankments. These embankments were built using locally derived sand and lightweight sawdust at two well characterised sites in southern Norway. Some conclusions from the work are as follows:

- o The peat layers are relatively thick (≈6 m) but are distinctly different at the two sites. At one site the peat has a low degree of decomposition and a high water content, whereas at the second site the peat is more decomposed with a lower water content.
- o The 2021 investigations at the two sites showed that on average the peat, near or beneath the embankments, had lower water content than that measured in 1963/64 probably due to embankment consolidation effects.

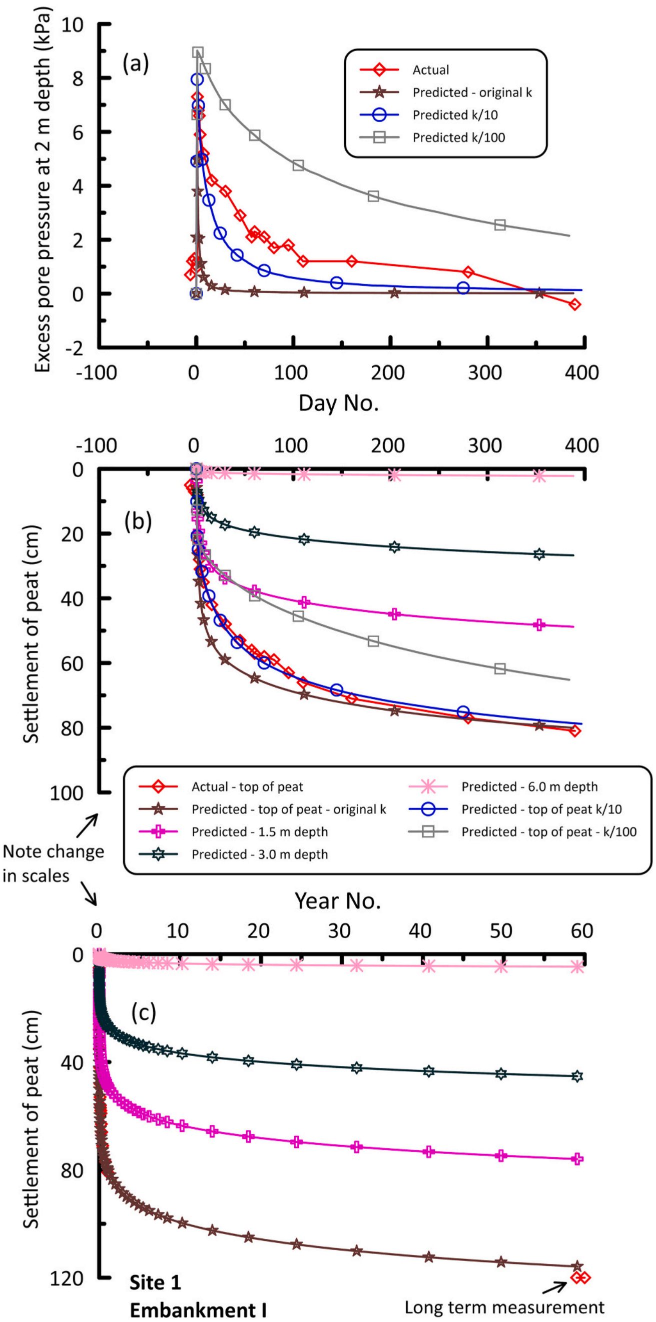

Fig. 13. Actual versus predicted measurements for Site 1 – Embankment I (a) excess pore water pressure, (b) settlement for first year after construction and (c) long term settlements.

Table 5
Summary of input parameters for Mohr Coulomb (MC) model for sand and sawdust.

Material	γ_{unsat} (kN/m^3)	γ_{sat} (kN/m^3)	e_0	E' (kPa)	ν_{ur} (−)	φ (deg.)	ψ (deg.)	c (kPa)	σ_t (kPa)	k_{v0} and k_{h0} (m/s)
Sawdust	6.7	11.5	2.25	1000	0.0	60	30	1.0	0	1×10^{-6}
Sand	17	20.7	0.6	20,000	0.3	45	15	5.0	0	Very high

Notes: Parameters are as given in Table 4 except: γ_{unsat}: unsaturated unit weight, E' = Young's modulus.

- o Peat properties were consistent with those published in the literature. The permeability of the peat is particularly difficult to estimate.
- o The engineering properties of the sawdust were studied in detail and in general the material behaved like a compressible fibrous peat.
- o The short-term behaviour of the embankment seems to have been controlled by the applied load, whereas in the long-term the more compressible peat showed most creep.
- o An important feature of the field behaviour was the unexpectedly high dissipation of the excess pore water pressures, which had been generated by embankment construction.
- o The embankments behaved as designed, i.e. the resulting settlements were such that the sawdust settled beneath the water table and subsequently remained submerged. The 2021 investigation showed that the sawdust remains in excellent condition.
- o Numerical modelling of one of the embankments was undertaken using the Soft Soil Creep constitutive model in the finite element code PLAXIS. Input parameters were chosen from the laboratory test results or from published correlations.
- o The predicted settlements, both long-term and short-term, matched the measurements very well. When the originally chosen permeability values were reduced the model was also able to capture the rapid dissipation of excess pore water pressure.
- o The good match between the numerical output and the field observations promotes confidence in the use of such modelling in future similar applications.

Ongoing work

These trial embankments have inspired the work currently ongoing in the EU Geolabs project "CLARIFIER" (https://project-geolab.eu). In this project large undisturbed blocks of peat from Norway were transported to the Netherlands. Model embankments, constructed using sand and sawdust, were placed on the peat and installed in the geotechnical centrifuge at Deltares. The embankments were then subjected to a variety of loading conditions with an aim to understand the failure mechanisms at play in this system.

CRediT authorship contribution statement

Michael Long: Writing – original draft. **Priscilla Paniagua:** Conceptualization, Project administration, Writing – review & editing. **Gustav Grimstad:** Data curation, Software. **Egil Berg Antoniazzi Sponås:** Investigation, Writing – review & editing, Methodology. **Endre Bjertness:** Investigation, Methodology, Writing – review & editing. **Stefan Ritter:** Conceptualization, Funding acquisition, Project administration, Supervision, Writing – review & editing.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

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

Acknowledgments

The authors would like to acknowledge the assistance of Nathan Townsend of NGI, who assisted in the 2021 field work at the two sites. Some of the testing of the sawdust was carried out at the laboratories of the Norwegian Public Roads Administration in Oslo by Marianne Dahl. Funding for the 2021 field investigation was partially provided by the Science Foundation Ireland (SFI) iCRAG Research Centre (https://icrag-centre.org).

References

Andresen, A., Kolstad, P., 1979. The NGI 54mm samplers for undisturbed sampling of clays and representative sampling of coarser materials. In: Proc., Proceedings International Symposium on Soil Sampling, pp. 13–21.
Bjertness, E., Sponås, E.B.A., 2021. Sawdust and wood chips in geotechnical engineering - Settlements of embankments on peat. Master's thesis in Civil and Environmental Engineering, Department of Civil and Environmental Engineering, Norwegian University of Science and Technology (NTNU), June.
Boussinesq, J., 1885. Applications des potentials à l'étude de l'équilibre et de mouvement des solides elastiques. Gauthier-Villars, Paris.
Carlsten, P., 1995. Construction methods for roads in peatlands areas. In: XI European Conference for Soil Mechanics and Foundation Engineering (ECSMFE). Danish Geotechnical Institute, Copenhagen, Denmark, pp. 8.13–18.18. DGF Bulletin 11.
Den Haan, E.J., Feddema, A., 2013. Deformation and strength of embankments on soft Dutch soil. Proc. Inst. Civil Eng. Geotech. Eng. 66 (3), 239–252.
Dykes, A.P., 2008. Tensile strength of peat: laboratory measurement and role in Irish blanket bog failures. Landslides 5 (4), 417–429.
Fadum, R.E., 1948. Influence values for estimating stresses in elastic foundations. In: 2nd International Conferecne on Soil Mechanics and Foundation Engineering (ICSMFE) Rotterdam, The Netherlands.
Flaate, K., Rygg, N.O., 1963. Sagflis i vegfyllig på myr (In Norwegian). Norsk Vegtidsskrift 12, 185–188.
Gautschi, M.A., 1967. Torf Als Baugrund (in German). Norwegian Geotechnical Institute (NGI). Report F.253.
Hanrahan, E.T., 1964. A road failure on peat. Géotechnique 14 (3), 185–202.
Hanrahan, E.T., Rogers, M.G., 1981. Road on peat: observations and design. Proc. Am. Soc. Civil Eng. J. Geotech. Eng. Div. 107 (GT10), 1403–1418.
Hobbs, N.B., 1986. Mire morphology and the properties and behaviour of some British and foreign peats. Q. J. Eng. Geol. Hydrogeol. 19 (1), 7–80.
ISSMFE, 1981. International manual for the sampling of soft cohesive materials. Published by Tokai Universtiy Press, Tokyo on belf of the International Society of Soil Mechanics and Foundation Engineering.
Jowsey, P.C., 1966. An improved peat sampler. New Phytol. 65 (2), 245–248.
Kallstenius, T., Bergau, W., 1961. Determination of horizontal ground movements. In: Proceedings 5th International Conference on Soil Mechanics and Foundation Engineering (ICSMFE) Paris, pp. 481–485.
Kværner, J., Snilsberg, P., 2008. The Romeriksporten railway tunnel — Drainage effects on peatlands in the lake Northern Puttjern area. Eng. Geol. 101, 75–88.
Lee, J.-S., Seo, S.-Y., Lee, C., 2015. Geotechnical and geophysical characteristics of muskeg samples from Alberta, Canada. Eng. Geol. 195, 135–141.
Long, M., Grimstad, G., Trafford, A., 2022a. Prediction of embankment settlement on Swedish peat using the Soft Soil Creep model. Inst. Civil Eng. J. Geotech. Eng. 175 (5), 507–522.
Long, M., Paniagua, P., Grimstad, G., Trafford, A., Degago, S.A., L'Heureux, J.-S., 2022b. Engineering properties of Norwegian peat for calculation of settlements. Eng. Geol. 308.
Lunne, T., Knudsen, S., Blaker, Ø., Vestgården, T., Powell, J.J.M., Wallace, C.F., Krogh, L., Thomsen, N.V., Yetginer, G., Ghanekar, R.K., 2019. Methods used to determine maximum and minimum dry unit weights of sand: is there a need for a new standard? Can. Geotech. J. 56 (4), 536–563.
Mesri, G., Ajlouni, M., 2007. Engineering properties of fibrous peats. J. Geotech. Geoenviron. Eng. ASCE 133 (7), 850–866.
Mesri, G., Godlewski, P.M., 1977. Time and stress compressibility interrelationship. J. Geotech. Eng. Div. ASCE 103 (5), 417–430.
NPRA, 2014. Statens Vegvesen Laboratorieundersøkelser Håndbok R210, Norway [in Norwegian].
O'Kelly, B., 2017. Measurement, interpretation and recommended use of laboratory strength properties of fibrous peat. Inst. Civil Eng. J. Geotech. Res. 4 (3), 136–171. Thomas Telford Ltd.
O'Riordan, N.J., Seaman, J.W., 1993. TRL Contractors Report 341, Highway Embankments over Soft Compressible Alluvial Deposits: Guidelines for Design and Construction. UK Transport Research Laboratory, Bracknell, Berkshire, UK.
PLAXIS, 2020. PLAXIS CONNECT Material Models, Version Edition. V20Plaxis, B.V, Delft, The Netherlands.
Sandbaekken, G., Berre, T., Lacasse, S., 1986. Oedometer testing at the Norwegian Geotechnical Institute. In: Yong, R.N., Townsend, F.C. (Eds.), Consolidation of Soils: Testing and Evaluation, ASTM STP 892. American Society for Testing and Materials, Philadelphia, pp. 329–353.
Vägverket, 1988. Bark och Flis some lättfylling i vägbankar - Publ 1986:77. Vägverket, Serviceavdelning Väg - och Brokonstruktion - Sektionen för geoteknik, Borlänge, Sweden.
Vesterberg, B., Carlsten, P., Lindh, P., 2016. Erfarenheter av byggmetoder på torvmark (in Swedish). Swedish Geotechnical Institute (SGI), Linköping, Sweden. Publication 26.
Waterman, D., Broere, W., 2005. Practical application of the Soft Soil Creep model-Part III. Plaxis Bull. Plaxis bv.