SURCHARGED EMBANKMENTS OVER PEAT IN NORTH-WEST IRELAND: PRIMARY CONSOLIDATION BEHAVIOUR

Farimah Fattahi Masrour[1], Bryan A. McCabe[1], Fintan J. Buggy[2], Paul Kissane[2] and Mike Long[3]

KEYWORDS

Primary consolidation, instrumentation, peat, surcharging

ABSTRACT

This paper presents selected data relating to the primary consolidation of blanket peat under surcharged embankments on a road improvement scheme in Co. Donegal, Ireland, completed in 2022. The back-calculated compression index was found to be compatible with values for Scandinavian peats and in keeping with a well-known empirical prediction at equivalent moisture contents. Coefficients of consolidation, derived using Asaoka final primary consolidation settlements as the starting point, exceeded those inferred from piezometer data, but both values significantly exceeded oedometer-derived values at equivalent effective stress levels. Carlsten's (1988) method produced a reasonable estimate of time to 90% consolidation once used within the intended loading range. A particular focus of the paper is on the corrections/assumptions necessary to interpret primary consolidation parameters in large strain scenarios.

1. INTRODUCTION

Peat's low undrained shear strength, high compressibility and long-term creep behaviour render construction in peatlands extremely challenging [1,2]. As a result, peat has traditionally been excavated and replaced with more competent material, especially on large road schemes. However, given the prevalence of peat in certain countries, e.g. 33.5% of the land area in Finland, 18% in Canada, 17.2% in Ireland and Sweden [3] and the important contribution that intact

[1] Civil Engineering, School of Engineering, University of Galway, Ireland; e-mail bryan.mccabe@universityofgalway.ie
[2] Roughan and O'Donovan Consulting Engineers, Sandyford, Dublin 18, Ireland
[3] School of Civil Engineering, University College Dublin, Dublin, Ireland

peatland ecosystems make to carbon sequestration [4], the excavation of large volumes of peat is nowadays considered to be unsustainable.

A recent road improvement project in a blanket peat area in north-west Ireland, in which the excavate-and-replace approach was not feasible due to ecological constraints, afforded the opportunity to consider surcharging as a means of limiting post-construction settlements arising from the construction of a multi-stage embankment. Surcharging is a well-established ground improvement practice in soft inorganic soils, but there is limited experience of its use in peat soils internationally. Transport Infrastructure Ireland [5] does not currently permit surcharging in soils with high organic contents such as peat but facilitated it on this project on an exceptional basis. A general overview of the scheme is presented elsewhere [6]. In this paper, primary consolidation data, derived from an instrument cluster at one cross section along the road alignment, are presented. These data are interpreted in the context of experiences in other Irish and Scandinavian peats, and Carlsten's [7] primary consolidation prediction method. The paper has a particular focus on the challenges associated with interpreting primary consolidation data in large strain scenarios, drawing on the experiences of Arulrajah *et al*. [8] who monitored a trial embankment on marine clay in Singapore.

2. PROJECT LOCATION AND SCOPE

The project involved the redevelopment of a 4.5 km section of the N56 national secondary route, north of Glenties, Co. Donegal, in northwest Ireland (Fig. 1).

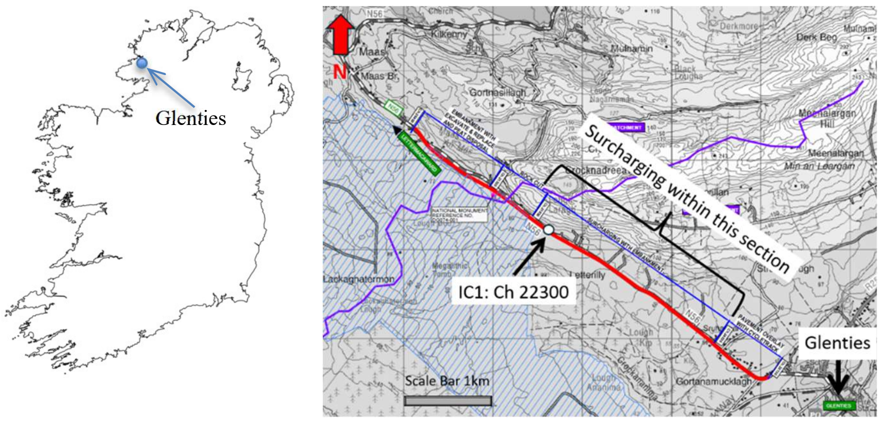


Fig. 1. (a) Glenties location; (b) plan of surcharging zones, IC1 location

A multi-stage surcharged embankment (Fig. 2a) was the preferred solution along ≈1.25 km of the road. In order to inform the embankment loading hold periods and assess embankment stability and peat settlements, six instrument clusters (ICs) were established along the road alignment. Each IC incorporated a foundation settlement plate (FSP), a subsurface profile gauge (SSPG) a vibrating wire piezometer installed near mid-depth within the peat, and an inclinometer pair, one on each side of the road (Fig. 2b). The data presented in this paper relates to IC1 at Chainage 22300 along the scheme (Fig. 1b).

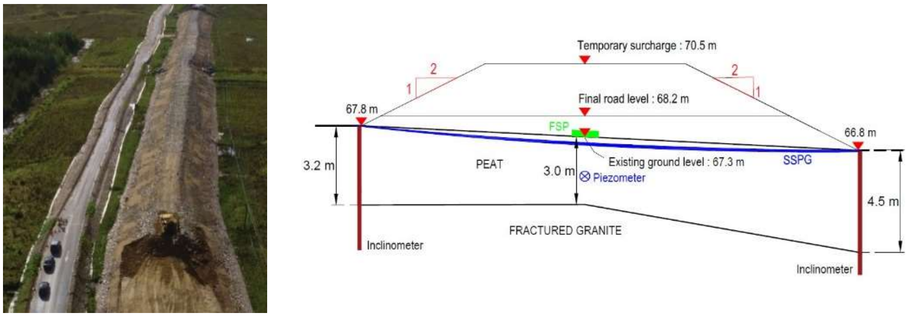


Fig. 2. (a) Surcharged embankment on N56; (b) Cross section schematic at IC1, including levels, peat thicknesses and instrumentation.

3. SITE CHARACTERISATION (AT IC1)

The following individual assessments of peat thickness were made at the centreline of the embankment at IC1: 3.5 m (gouge auger), 2.7 m (Russian sampler), 2.8 m (CPTu) and 3.3 m (CPT ball); an average peat thickness at the centreline of 3.0 m was adopted for subsequent interpretation (Fig. 2b). Peat thicknesses of 4.5 m and 3.2 m were inferred from inclinometers at the west and east sides respectively. The Russian sampler log indicated the existence of fractured granite beneath the peat, while the CPTu ball identified a thin sand layer 100 mm thick, both suggesting the potential for double drainage.

According to the Von Post [9] classification system, the peat at IC1 was categorised as H4-H7, with the degree of humification generally increasing with depth (Fig. 3a). It consisted of both fine and coarse fibres, with timber noted in the lower half. A broad spectrum of moisture contents (w) in the range 695-1400% was observed (average water content ≈980%), without an apparent trend with depth (Fig. 3b). CPT ball end resistance (q_{ball}) and pore water pressure measured at the shoulder position (u_2) are shown in Fig. 3c.

Three different methods were employed to determine the undrained shear strength (c_u) of the peat. A single direct simple shear (DSS) test from a specimen obtained from a block sample near IC1, consolidated to a normal stress of 7 kPa, gave c_u=6 kPa. In addition, c_u was estimated from a correlation with shear wave velocity (V_s) (also plotted in Fig. 3b) and w, developed by Trafford and Long [10] from Irish, Scottish and Dutch peat sites (Eqn. 1):

$$c_u = 55.8\left(\frac{V_s}{w}\right)^{0.683} \qquad (1)$$

This approach yielded c_u values in the range 3-5 kPa. A separate assessment of c_u was made from CPT ball end resistances adopting $N_{t,ball} = 15$ [6], yielding values typically between 4-7 kPa.

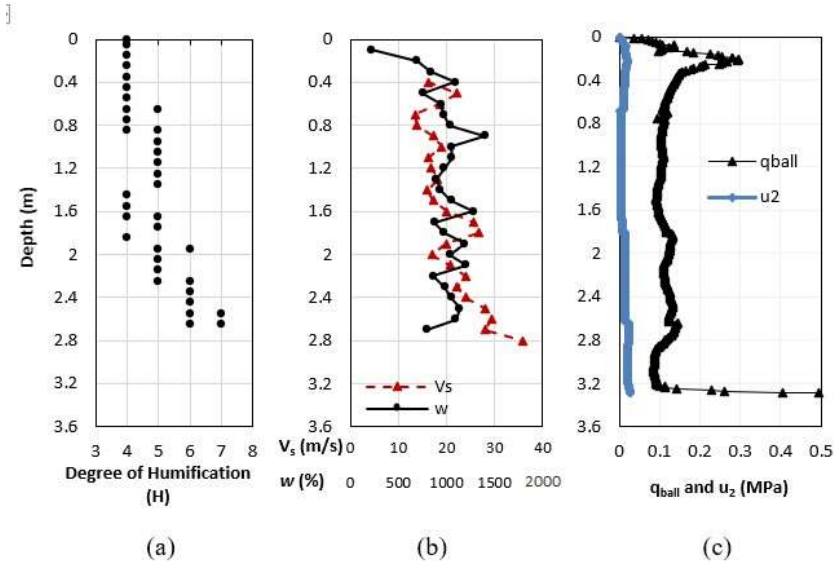


Fig. 3. Variation of (a) degree of humification, (b) water content (w) and shear wave velocity (V_s) and (c) CPT ball data with depth within peat

Six maintained load oedometer tests were performed on 80 mm diameter, 19 mm high peat specimens (from multiple locations) obtained from serrated-edge, thin-walled tubes. Sample depths ranged from 0.5-2.4 m and natural moisture contents from 520% to 1136%. The 24 h load stages were 2.5, 5, 7.5, 10, 20, 40, 80, 120 and 80 kPa. Two tests showed evidence of a yield pressure in the peat of between 5 and 10 kPa. Coefficients of consolidation (c_v) typically varied between 2-5 m^2/yr at initial load stages, reducing to 0.2-0.6 m^2/yr above 40 kPa.

4. STAGED EMBANKMENT CONSTRUCTION

The embankment at the location of IC1 was built in eight stages, each 0.5-1.0 m in height with a minimum hold period of 2 weeks, corresponding to an average filling rate of ≈0.14 m/week [6]. A minimum hold period of 90 days at maximum fill height was specified in order to achieve a degree of dissipation (U_v) of at least 90%. At IC1, the maximum fill height of 4.6 m (including the fill displaced below the original ground level due to settlement) was maintained for 99 days prior to surcharge release to 2.2 m (Fig. 2b).

Fig. 4 illustrates the evolution of vertical total stress with time at IC1 due to embankment loading, calculated (for consistency) at the current level of the piezometer (initially 1.75 m below original ground level). In keeping with recommendations by Arulrajah *et al.* [8], this vertical total stress was corrected to allow for submergence of a significant portion of the embankment fill beneath the groundwater table (which was ≈0.6 m deep at IC1). The corrected total stresses are based on a measured fill dry density (γ_d) of 20.8 kN/m^3 (determined by *in situ* nuclear density meter), with the saturated density calculated using a specific gravity of 2.6 for the (largely) sandstone fill used.

5. SETTLEMENT MONITORING

General

The peat settlement arising from staged embankment loading, derived from both the FSP and SSPG (at the centre of the embankment), is also presented in Fig. 4. During the early stages, the initially rapid rate of settlement was observed to diminish significantly over the hold period. From Stage 4 onwards, the settlement-time trend for each stage became more uniform, which may be attributed to the cumulative effects of consolidation from previous load increments, and to the significant reduction in c_v with increasing vertical effective stress levels characteristic of peat [11,12]. The FSP and SSPG settlements (at the centre of the embankment in the latter case) of 1.39 m and 1.25 m respectively correspond reasonably well given the different measurement approaches involved (the SSPG tube was embedded in compacted gravel). Immediately prior to surcharge release, these settlements corresponded to vertical strains of ≈46% and ≈41% respectively. Some rebound in both settlement readings is observed upon release of the surcharge.

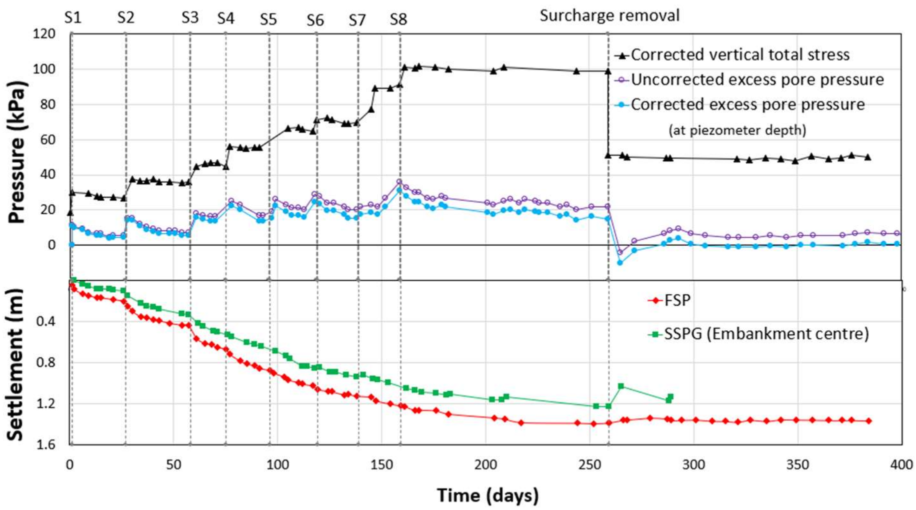


Fig. 4. Corrected total stress, uncorrected and corrected excess pore water pressure and FSP and SSPG (central) settlements due to multi-stage embankment loading

By way of comparison, Rodgers [13] stated that strains of 40-50% (assumed by the authors to exclude creep) are typical in normally consolidated (NC) Irish peats with moisture contents of 1000-1500% subjected to 2 m high embankments (40 kPa applied pressure). At IC1, equivalent strains required much higher applied pressures. Carlsten's [12] method for estimating relative compression (strain) from water content and applied loading, the basic version of which assumes peat to be normally consolidated, predicts ≈55% strain after Stage 8. The method accommodates light overconsolidation by correcting the applied stress by subtracting the difference between the yield pressure and *in situ* vertical effective stress. Even with a correction as high as 10 kPa, the effect is merely 2-3%, so the method still overpredicts the measured strains.

The transverse surface settlement profile was assessed using the SSPG data. The profiles shown in Fig. 5 reflect the settlement at the end of each loading stage. The profile is relatively symettrical, which is somewhat surprising given that gouge augers and inclinometers indicated a greater thickness of peat to the west of the embankment (Fig. 2b). The new road alignment did not encroach upon the previous alignment at this location.

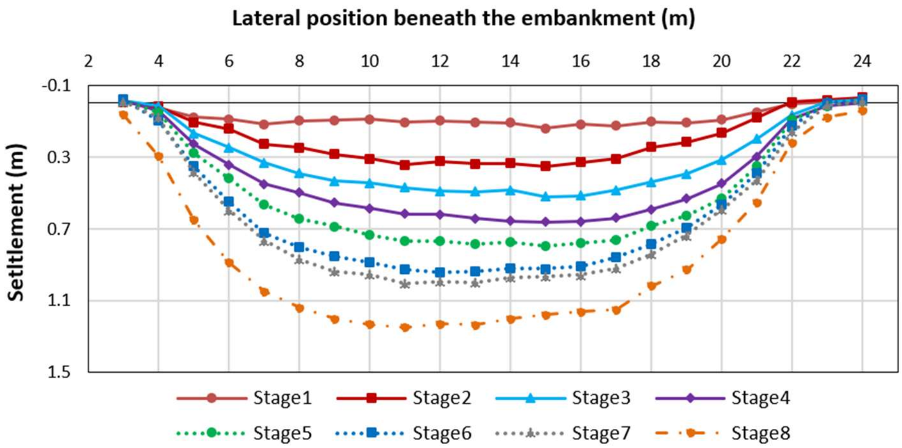
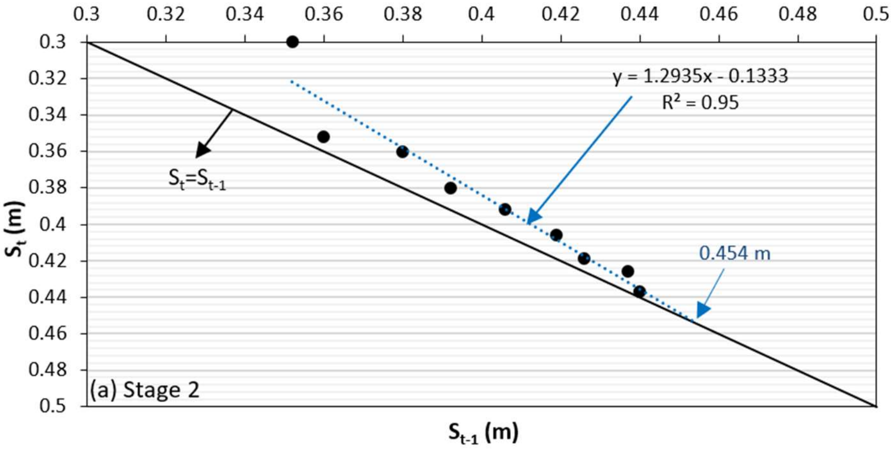


Fig. 5. SSPG settlement profiles at the end of stages 1-8 at IC1

Asaoka method and coefficient of consolidation

Given the greater frequency of the FSP data than the SSPG data, the Asaoka [14] method was applied to the former to estimate the final primary consolidation settlement at IC1. The method is applicable to situations where the load is constant and for which $U_v > 60\%$. Only stages 2 and 8 met the latter criterion (Fig. 4). Also, because the Asaoka method requires a consistent data frequency and the data were captured at irregular intervals, the settlement dataset was updated, using a linear interpolation approach, to achieve equal time intervals of ≈1 week. Arulrajah *et al.* [8] studied the effect of time interval on Asaoka predictions, and recommended a maximum interval which corresponded to 3% of the time to achieve 90% consolidation (t_{90}) within the marine clay. For IC1, the measured t_{90} = 96 days for stage 8 loading suggests a minimum recommended frequency of 3 days. The error in using 1 week intervals herein is not believed to be significant, based on their findings.

The Asaoka plots for stages 2 and 8 are shown in Fig. 6, with predicted final primary consolidation settlements of ≈0.454 m and ≈1.43 m respectively, inferred from the points of intersection of the two lines. The actual settlement at the end of stage 2 of ≈0.44 m and the settlement immediately before surcharge release of ≈1.39 m in Fig. 6, suggest that $U_v \approx$ 94% and 97% for stages 2 and 8, respectively. An important inference from Arulrajah *et al.* [8] is that the settlement datasets reflecting high degrees of consolidation, as is the case here, will lead to more accurate final predicted settlements.



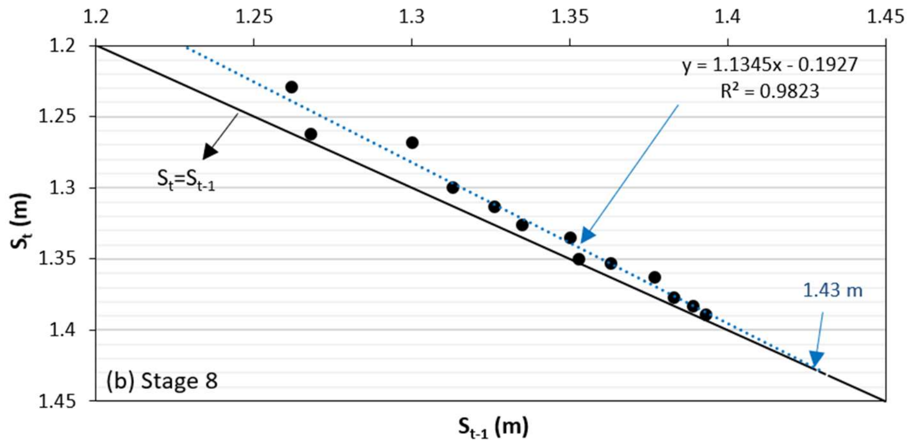


Fig. 6. Asaoka primary settlement plots for (a) Stage 2 and (b) Stage 8.

The entire embankment construction at IC1 took almost 163 days from the start of loading to the time at which its maximum fill level was achieved in stage 8. Using the Asaoka predicted final settlement of ≈1.43 m, a global c_v value (corresponding to a vertical effective stress increase from 7 to 82 kPa) was calculated by assuming that the full embankment height was attained in an equivalent single stage at the midpoint (81 days after commencement) of the actual loading period. Back-calculated values of c_v were determined based on (i) the value U_v =97% inferred from settlement data, and (ii) the interpolated times to achieve U_v =90%, considering the difficulty in calculating the time factor T_v value reliably for U_v values approaching 100%.

Assuming double drainage of the peat, the drainage path length (*d*) for the entire embankment loading was calculated in two ways. The first method adopted an average layer thickness, i.e. d = [3-(1.39/2)]/2=1.15 m. Acknowledging that the load was assumed to be applied at 81 days, the second method used an average of the settlements at the midpoint of loading (0.78 m) and the end of the primary settlement (1.39 m), resulting in: d = (3-1.08)/2=0.96 m. Corresponding c_v values based on U_v=90% are 4.3 and 2.9 m²/year, respectively. Values based on U_v=97% are 3.8 m²/year and 2.6 m²/year respectively.

The same calculation was conducted to determine the c_v value for stage 2. The corresponding vertical effective stress range (13-19 kPa) exceeds the peat's yield stress (σ'_{vy}) of ≈10 kPa estimated using $\sigma'_{vy} = 150/e_0$ [1] (with e_0 estimated from the average water content ≈980% and G_s=1.5 [15] assuming saturated conditions) and measured in two of the oedometer tests. Two U_v values (90% and 94%) and two drainage paths were considered based on the aforementioned logic. The c_v values were calculated as 30.8 m^2/yr and 33.2 m^2/yr (d = 1.34 m), and 23.5 m^2/yr and 25.3 m^2/yr (d = 1.39 m), for U_v=90% and U_v=94% respectively. These higher back-calculated c_v values reflect peat properties during an earlier stage in the loading history, when effective stresses were lower.

Carlsten's [12] method produced a reasonable estimate of t_{90} for stage 2 (25 days, versus 27 days observed in the field), but significantly underestimates t_{90} for the entire embankment (50 days, compared to 96 days in the field). The latter discrepancy may be the result of an attempt to extrapolate the method to applied pressures well in excess of the intended upper limit (50 kPa).

Compression index

Equation (2), used to back-calculate the compression index (C_c), was conservatively applied from Stage 3 to Stage 8 to guarantee that the settlement (ΔH) pertained to the NC state only.

$$\frac{\Delta H}{H_0} = \frac{C_c}{1+e_0} \log\left(\frac{\sigma'_{vS}}{\sigma'_{v2}}\right) \qquad (2)$$

where σ'_{v2} is the effective stress at the end of Stage 2, σ'_{vs} is the maximum effective stress at the end of primary consolidation (as inferred from the Asaoka method). H_0 was taken as the layer thickness at the end of stage 2 (3-0.44 ≈2.6m). This yielded a C_c value of 9.1, tallying well with the estimate C_c= w/100 proposed by Mesri and Ajlouni [15] for fibrous peat. By way of comparison, Carlsten [7] and Long *et al.* [16] report value in the range 6-8 and 6-9 for Swedish and Norwegian peats respectively, for w≈1000%.

6. PORE PRESSURES AND COEFFICIENT OF CONSOLIDATION

The piezometer was installed at a depth of 1.75 m within the peat layer, slightly below the assumed mid-depth, directly beneath the foundation settlement plate. In interpreting pore pressures, it was necessary to allow for a reducing instrument position over time as it settled in tandem with the peat, as highlighted by Arulrajah *et al*. [8]. In the absence of this adjustment, the interpreted excess pore pressures would exceed the actual values, leading to an underestimation of U_v. Given the relatively uniform peat consistency with depth (uniform profiles of w and V_s in Fig. 2b), and a very modest decay with depth (over the 3.0 m peat thickness) of imposed total stress from the embankment (calculated using Boussinesq theory), its level was corrected by ≈42% (i.e. [3 - 1.75] m/ 3 m) of the surface settlement of the peat at any time. The excess pore pressure data, corrected on this basis, are included in Fig. 4.

The corrected excess pore water pressure exhibited a sharp initial increase during the earliest part of each load increment, followed by a gradual decline until the end of the loading stage, associated with continuous soil compression. After removing the maximum surcharge, a temporary negative corrected excess pore water pressure was measured for about 3 weeks, related to the swelling of peat, before reverting to zero.

The number of data points and/or the degree of dissipation was insufficient to make separate estimates of c_v for each of the stages; with settlement-log time graphs generally linear. However, a global estimate of c_v was made from the corrected excess pore pressures, assuming as before that the maximum load was applied in a single increment at the midpoint of staged construction. The calculation of U_v using this approach required an estimate of the excess pore water pressure that would have arisen had the load been applied in a single increment. To this end, the increases in excess corrected pore pressure (Δu) corresponding to the increases in corrected total stress ($\Delta\sigma$) are compared, at the outset of each stage, in Fig. 7. The data for stage 7 were difficult to interpret and were excluded. Both values are in reasonably good agreement; Δu falls slightly below $\Delta\sigma$ for five of the stages but this may be a result of a lag between filling and the first pore pressure measurement, in which case the peak Δu is likely to have been missed. On the basis of Fig. 7, it was deemed reasonable to assume that the excess corrected pore pressure increment corresponding to a single stage corresponds approximately to the corrected total stress increment of 90 kPa (i.e. assuming Skempton's pore pressure coefficient A≈1).

Arulrajah *et al.* [8] also highlighted the need to adjust U_v corresponding to an individual piezometer depth to reflect the mean level of dissipation over the entire layer. A standard double-drainage isochrone, based on Terzaghi's theory of primary consolidation, was used to effect the adjustment. Again considering the two possible values of d, the average U_v for the whole peat layer was calculated as 88%, with corresponding c_v ≈1.52 m^2/year (d=0.96 m) and 2.2 m^2/year (d=1.15 m). These values are lower than the Asaoka-derived values, an outcome also reported by Arulrajah *et al.* [8], who attributed this to the stress-strain non-linearity of the soil. It is clear that there are many more assumptions/corrections involved in the derivation of c_v from pore pressures, rendering the Asoaka approach preferable where the data requirements are met.

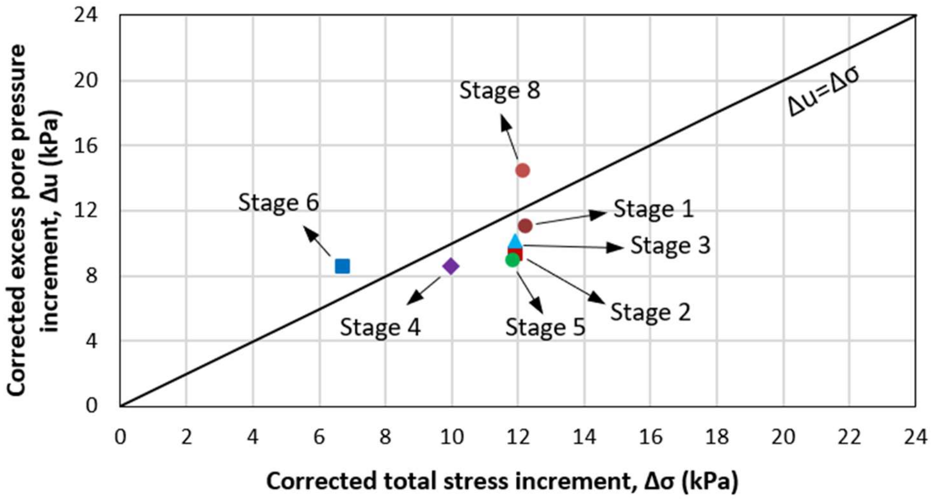


Figure 7. Variation of pore water pressure changes corresponding to the corrected total stress at IC1

Field values of c_v (derived from either the Asaoka method or pore pressures) significantly exceed the oedometer values. The field c_v value for stage 2 of 23.5-33.2 m^2/year exceeds the oedometer range 2-5 m^2/year by an order of magnitude at low stresses, while the values at higher stresses, (c_v = 1.52 - 4.3 m^2/year, depending on interpretation) also exceed the oedometer values of 0.2 - 0.6 m^2/year. This indicates that the oedometer values may significantly overestimate the duration for a given extent of peat consolidation in the field.

7. CONCLUSIONS

Primary consolidation parameters for a blanket bog peat have been back-figured from settlements and pore pressures measured at one location during the construction of a multi-staged embankment in north-west Ireland. The following conclusions have been drawn (for the IC1 location):

- Peat strains in excess of 40% were recorded as a result of a vertical effective stress increase of 82 kPa; these were lower than predicted by the Carlsten [12] method.
- The back-calculated compression index corresponds well with that measured for Norwegian and Swedish peats at corrresponding *in situ* water contents, and is also predicted well by the well-known Mesri and Ajlouni [15] correlation.
- Asoaka-derived c_v values exceed those derived from piezometer data, in keeping a comparison of both methodologies at a marine clay site in Singapore [8]. However, the number of assumptions necessary to derive c_v in the latter case, as highlighted in this paper, is believed to render it an inferior approach. A key uncertainty in relation to the position of a piezometer within a layer subject to large strains could be removed by installing a companion extensometer. The Carlsten [12] method appears to predict t_{90} reasonably well for pressures below its 50 kPa limit of applicability, but may underpredict t_{90} at higher pressures.
- Field values of c_v exceed laboratory values significantly at equivalent stress levels. As a result, designers using laboratory c_v values may recommend longer staged embankment hold periods than are actually necessary, unless performance-monitoring instrumentation is used. In future, piezocone or piezoball dissipation tests may serve as a better means of assessing excess pore pressure dissipation rates.

Primary consolidation parameters for all six ICs will be presented in a future paper. In addition, the potential of surcharging to reduce long-term creep settlements is being assessed by the University of Galway, through long-term monitoring of the FSPs and SSPGs at the N56 site, in conjunction with numerical modelling and a programme of oedometer testing on peat, both modelling the surcharging process.

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REFERENCES

[1] M.A. Ajlouni: Geotechnical properties of peat and related engineering problems. PhD thesis, University of Illinois, Urbana-Champaign, 2000.

[2] A.R. Duggan, et al.: An embodied carbon and embodied energy appraisal of a section of Irish motorway constructed in peatlands. Construction and Building Materials, Vol. 79, pp. 402-419, 2015.

[3] E.R. Farrell: Chapter 35: Organics/peat soils. In ICE manual of geotechnical engineering, pp. 463-479. Thomas Telford Ltd, 2012.

[4] J. Beaulne, et al.: Peat deposits store more carbon than trees in forested peatlands of the boreal biome. Scientific Reports, Vol. 11.1, pp. 1-12, 2021.

[5] Transport Infrastructure Ireland: Specification for road works series 600-Earthworks (including erratum No.1), June, 2013.

[6] P. Kissane, et al.: Staged construction of surcharged embankments over peat for a national road in Co. Donegal, Ireland. Proceedings of the XVIII European Conference on Soil Mechanics and Geotechnical Engineering, 2024.

[7] P. Carlsten: Geotechnical properties of some Swedish peats. NGM 2000, XIII Nordisk Geotechnikermotet, Vol. 5-7, No. 6, pp. 51-60, 2000.

[8] A. Arulrajah, et al.: Factors affecting field instrumentation assessment of marine clay treated with prefabricated vertical drains. Geotextiles and Geomembranes Vol. 22, No. 5: pp. 415-437, 2004.

[9] L. Von Post: Sveriges Geologiska Undersöknings torvinven:: tering och några av dess hittills vunna resultat, Sr. Mosskulturfor 1:1–27

[10] A. Trafford, M. Long: Relationship between shear-wave velocity and undrained shear strength of peat, ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 146, No. 7, pp. 1-10, 2020.

[11] L. Samson, P.L. Rochelle: Design and performance of an expressway constructed over peat by preloading. Canadian Geotechnical Journal, Vol. 9, No. 4, pp. 447-466, 1972.

[12] P. Carlsten: The use of preloading when building roads in peat. Proceedings of the 2nd Baltic Conference on Soil Mechanics and Foundation Engineering, pp. 135-143. 1988.

[13] M. Rodgers: Construction of embankments at soft soil sites, Proceedings of Seminar on road embankments on soft ground, The Institute of Engineers of Ireland, pp. 72-77, 1996.

[14] A. Asaoka: Observational procedure of settlement prediction, Soils and Foundations, Vol. 18, No. 4, pp. 87-101, 1978

[15] G. Mesri, M. Ajlouni: Engineering Properties of Fibrous Peats. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 133, No. 7, pp.850-866, 2007.

[16] M. Long, et al.: Engineering properties of Norwegian peat for calculation of settlements. Engineering Geology, Vol. 308, p. 106799, 2022.