Deformation performance and stability control of embankments on peat soils

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Abstract. Embankments constructed on very soft soils such as peat are frequently controlled by means of an observational method approach. Concurrent observations of maximum settlement and maximum lateral movement of the soil profile at the embankment toe can be used to calculate a deformation ratio which varies with load application and can be interpreted as an indicator of stability. Trigger or threshold values have been suggested for single stage embankments constructed on soft mineral soils, but little guidance is available for multi-stage embankments on organic, peat soils. This research is a compilation of analysis of several published international case histories involving instrumented embankments and load tests constructed on peat soils. Suggested limits for threshold values are developed for deformation ratios and shear strain rate. Recommendations for instrumentation layout, monitoring frequencies, data interpretation are developed. The goal of the paper is to summarise published experience and provide guidance to practitioners using the observational method to control stability of embankments on very soft peat soils.

1 Introduction

Matsuo & Kawamura [1] published graphs for estimating stability of multi-stage embankments on soft clays derived from a ratio of deformation measurements δ/d. Lateral deformation at the ground surface δ was defined at the toe of the embankment and settlement beneath the embankment centreline is d. δ is not the same as the maximum of the lateral deformation profile beneath the toe $D_{max.}$ as adopted in this study. The authors acknowledged that the maximum settlement S_{max} observed anywhere beneath the embankment crest and the maximum lateral deformation D_{max} beneath the toe are inherently better indicators of potential failure.

The variation of δ/d plotted against d are presented in Figure 1 for field cases including a theoretical failure threshold, conservatively developed assuming undrained behaviour without consolidation or strength gain. Approaching this failure line essentially indicates that horizontal plastic flow due to shear strain is large compared to vertical settlement and consequently the factor of safety is low. Deformation ratios were reported to be strongly influenced by the ratio of embankment width to soft layer thickness. Tsukisappu was constructed rapidly at a filling rate of 1.4m/week on a 6m depth of soft peat and failed at 4m height.

Tavenas & Leroueil [2] developed a framework of expected observed field behaviour for maximum lateral toe deformation D_{max} and centreline settlement S for normally and over consolidated clays. Leroueil & Rowe [3] describe peat responses to embankment loading.

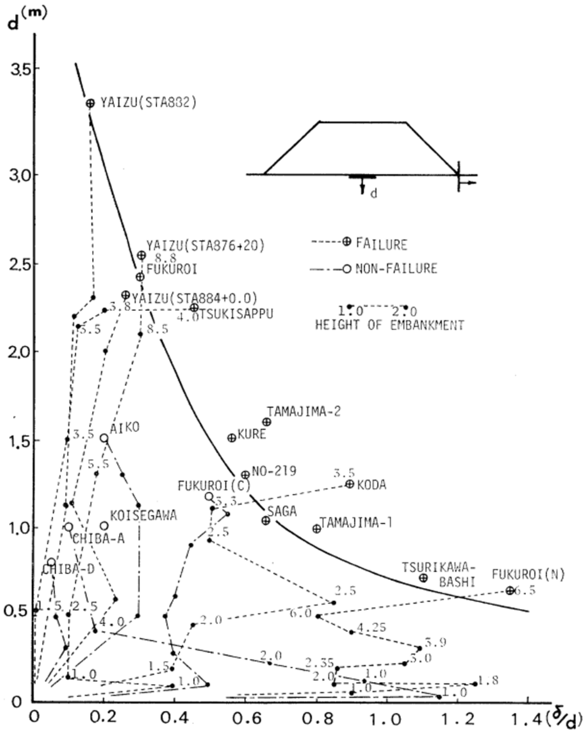

Fig. 1. Deformation ratios δ/d vs settlement d. Matsuo & Kawamura [1].

High initial permeability of peat is expected to result in partially drained conditions upon the application of an embankment load. Slower construction rates of 0.15 to 0.3 m fill height /week were observed to result in stability for embankments on San Francisco Bay peat up to 3m height, whereas faster rates of filling around 0.9 m/week resulted in failure at 2.4m height. Primary

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consolidation durations are estimated to take from 5 to 200 days in peat deposits from 2m to 10m thick.

Jardine [4] suggested the following observational limit criteria for Factor of Safety = 1.2 for single stage embankments on typical soft clay soils.

- D_{max} < 80mm for embankment height H < 7m; D_{max} < 120mm (toe berms or flat side slopes);
- Max shear strain γ_{vh} = 2%
- Max shear strain rate $\delta \gamma_{vh} / \delta t$ = 0.2% / day

Suggested deformation ratio limit criteria for multi-stage embankments on soft mineral soils, (excluding lean clays and peat) $\Delta D_{max} / \Delta S_{max} < 0.3$.

Buggy [5] examined filling rates and maximum observed deformation ratios for large road embankments in Ireland constructed using multi-stage construction on soft clay / silt soils and concluded a limit peak deformation ratio of 0.6 was acceptable for stability. Peak deformation ratios typically occur during first stage filling at embankment heights < 4m and failures were often related to proximity to creek banks, ditches and excessive filling rates typically > 1m/ week.

2 Case histories

Deformation and pore pressure data was obtained for a variety of multi-stage embankments constructed on peat soils and summaries are presented in subsections below. Projects selected all have frequent pore pressure and deformation measurements as well as load histories plus well documented ground conditions. Tables 1 & 2 indicate relevant characteristics of peat or organic soils, embankment geometry, geosynthetic reinforcement and loading rates where known.

Table 1. Peat properties.

Location	Peat Depth (m)	Moisture Content (%)	Von Post H / Humificatio n R (%) / LOI N (%)	C_{uo} (kPa) *q_c (MPa)*
Mukasa, Japan	47 - 54	150 - 500	-	-
Toubetsu, Japan (peat / org clay)	4.8 / 3.15	281–814 / 73–224	H3 – H4	*0.12 - 0.35 / 0.17- 0.35*
Antoniny, Poland (peat / calcareous)	3.1 / 4.7	310 – 315 / 105 - 114	R = 50-70	5 –8[1] *0.1 - 0.2*
Uitdam, Netherlands	5	645 - 1240	H2 – H3; N = 75-92	*0.1 – 0.15*
Glenties, Ireland	2.4 – 3.8	800 - 1240	H3 – H7 N = 64 -98	3 – 6[2]

[1] C_{uo} from corrected field vane and triaxial lab tests
[2] C_{uo} from DSS, q_{ball}, in situ shear wave velocity v_s

Table 2. Embankment geometry and loading stages and peat layer drainage boundary conditions.

Location	Width / Side Slope (H:V)	Max Fill Height[1] (m / *kPa*)	Number of stages	Peat / Soft layer Drainage Path Boundary
Mukasa, Japan	85 / 4:1 (berms)	15	2	single
Toubetsu, Japan	30 / 1:1	8	14	double
Antoniny, Poland	35 / 3:1	5.8	3	double
Uitdam, Netherlands	7 / N.A.	*71.9 kPa*	5	single
Glenties, Ireland[2]	20 – 22 / 2:1	4.7 – 5.2	8	double

[1] Fill height includes settlement below ground level
[2] Basal reinforcement 400 kN/m ultimate tensile strength

2.1 Mukasa, Japan

The Mukasa test embankment was constructed over 50m depth of alluvial or diluvial peat layers (Apt and Dpt), interbedded with clay and sand layers (Ac and As) and underlain by a diluvial gravel (Dg) shown in Figure 2. A slip failure did not occur during construction, but peat layers below 30m depth experienced unexpected large compression. This is believed to be the result of high excess pore pressures caused by artesian conditions in the underlying gravel layer and relatively high depositional rates. Mean natural moisture content within the peat sublayers prior to embankment construction ranged from 150% to 500 % and corresponding bulk density from 2.6 to 1.8 t/m^3. Much of the material might be classified as highly organic clay with the deeper layers Apt5, Apt6 and Apt8 exhibiting the higher moisture contents above 300%.

The 85m wide, 150m long embankment was divided into three 50m long sections with ground improvement using cardboard (CBD) or sand (SD) vertical drains installed to 20m depth in two sections and no improvement in a third section. The loading and settlement history is shown in Figure 3. The cross section included multiple toe berms and was constructed in two main stages over a period of 10 months to achieve a 7.2m height above original ground level and an aggregate fill thickness of 15m. Average filling rates in load stages 1 and 2 were 0.19 and 0.21 m/week respectively. Following a two year hold period a third stage was added to replace fill lost to settlement.

Some limited inclinometer data was published and shown in Figure 4. The largest lateral deformations of 1.7m and 1.45m occurred on the left side of the embankment with local maxima evident at depths of 7m and 25m located in soft clay Ac1 and peat Apt5 layers

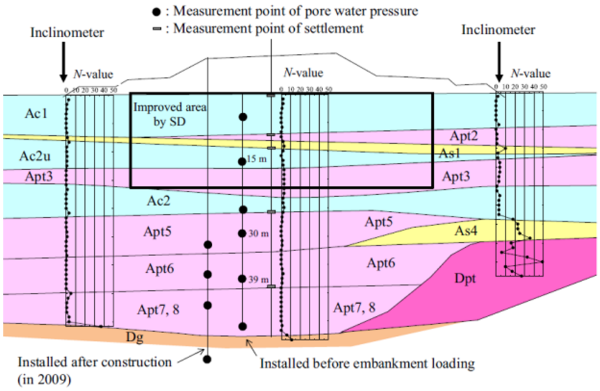

Fig. 2. Mukasa test embankment SD cross section with instrumentation locations. Tashiro et al [6].

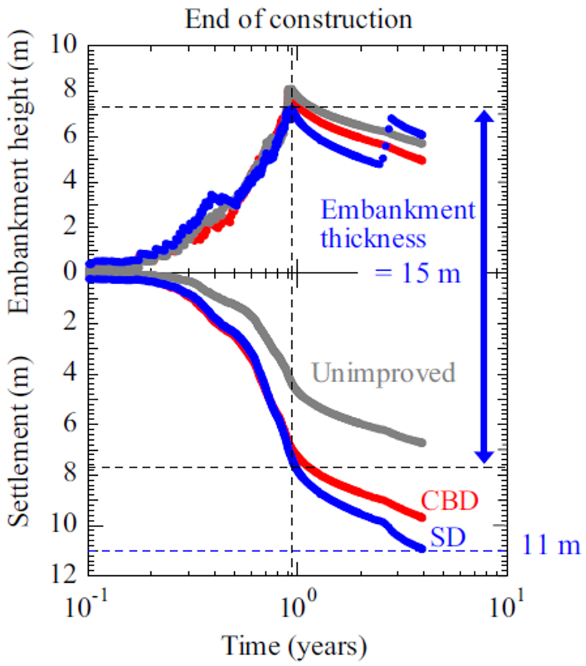

Fig. 3. Mukasa loading and settlement history for three embankment zones. Tashiro et al [6].

respectively. Profile gauge data confirmed that the maximum settlements occurred at the embankment centreline. A summary of the deformation ratios for the peat layers below 12m depth on the more critical left side of the embankment is provided in Table 3. The inclinometer observation dates do not coincide with the peak loading in stages 1 and 2 (days 142 and 345). The maximum lateral deformation profile could also be located at the internal berm toe, just outside the limits of sand drain improvement, rather than at the actual inclinometer position at the outermost toe shown in Figure 2.

Table 3. Deformation ratios in peat at Mukasa (left side).

Time t (days)	Load Stage	Embankment Ht H (m)	Centreline Settlement $S_{<As1}$ (m)	Max Lateral Displacement $Y_{max<As1}$ (m)	Deformation Ratio <As1 $Y_{max/S<As1}$
119	1	2.3	0.50	0.07	0.14
188	1 (hold)	2.9	1.20	0.42	0.35
302	2	6.0	3.74	1.13	0.30
364	2 (hold)	6.8	5.29	1.45	0.27

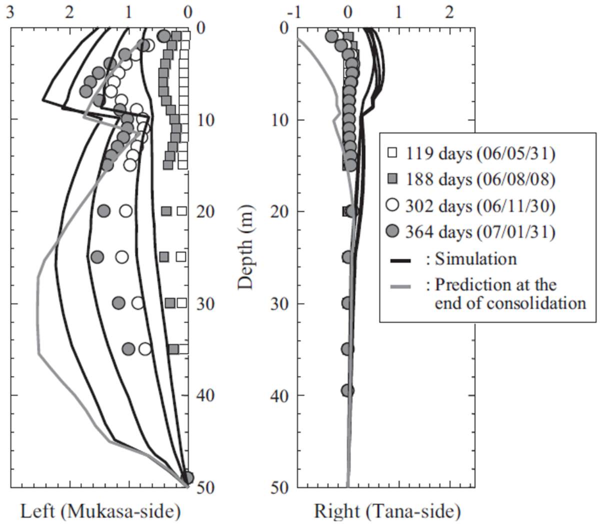

Fig. 4. Mukasa test embankment inclinometer profiles. Tashiro et al [6].

2.2 Toubetsu, Japan

A trial embankment was successfully constructed in 8.5 months at Toubetsu expressway, Hokkaido, Japan. Figure 5 shows the cross-section of the trial embankment and soil profile which consisted of peat, underlain by a soft organic clay and fine sand. A sand blanket 0.75m thick was initially placed and held for a period of 12 weeks. Subsequently 4.25m embankment fill was placed in 14 incremental stages over a period of 22 weeks. Stages 1 – 7 were constructed at an average filling rate of 0.35 m/week, followed by stages 8 – 14 at an average of 0.13 m/week. Graphs of the fill height above ground level, gross settlement and maximum lateral deformation v time are presented in Figure 6. A maximum settlement of 3m and lateral deformation of 0.63m was observed at load stages 10 to 14.

Deformation ratio v time is presented in Figure 7. The "total" ratio includes settlement from the organic clay and deeper layers and is a better representation of compressive and shear deformations from both contributing soft soil layers. Deformation ratios increase steadily during initial load stages 1 – 8 up to a maximum of 0.43 before declining to 0.23 at the final load stage 14. Lateral deformation profiles indicate that the maximum deformation consistently occurs near the base of the peat layer at around 4m depth. No measurements of lateral deformation were made immediately following construction of the sand blanket.

Excess pore pressures measured in the peat layer midpoint beneath the embankment centre are presented in Figure 8. No data was obtained during sand blanket installation, but subsequent readings show an increase to a peak of 42 kPa from stage 1 up to stage 9 hold, close to the peak deformation ratio, before declining to 23 kPa in stage 14. Excess pore pressures observed in the organic clay layer are very similar. Partial consolidation in the range of 41% to 78 % at peat mid layer depth can be inferred from the measured excess pore pressures as a ratio of total stress applied in stages 9 and 14 respectively. Total stress loading applied includes additional fill depth due to settlement and the buoyancy effect of fill below the 0.4m ambient water table depth.

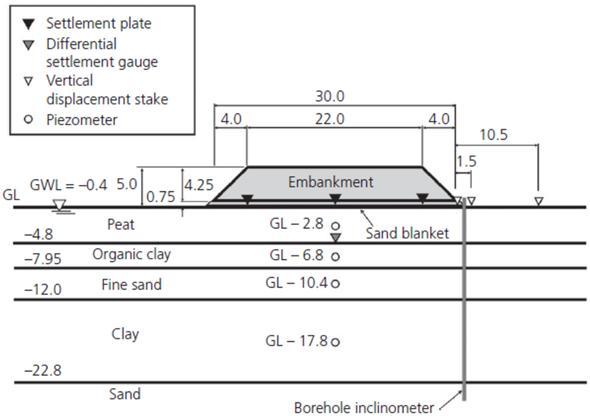

Fig. 5. Toubetsu test embankment cross section with instrumentation locations. Hayashi & Yamazoe [7].

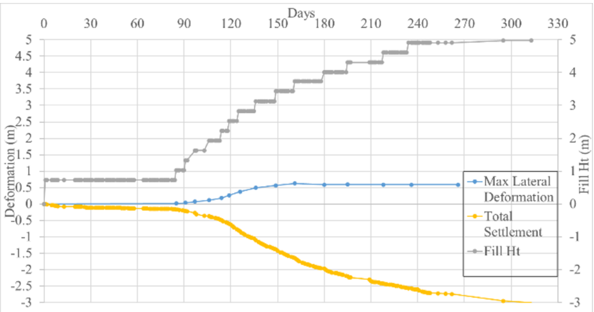

Fig. 6. Toubetsu settlement, maximum lateral deformations and fill height vs time.

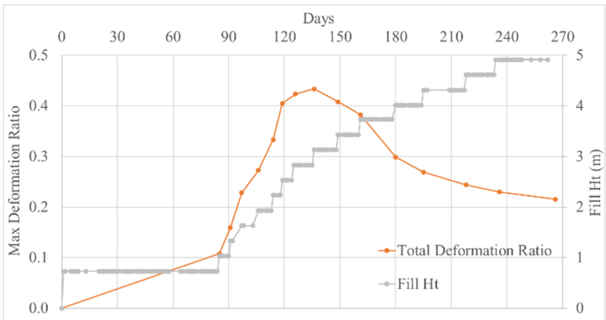

Fig. 7. Toubetsu deformation ratios and fill height vs time.

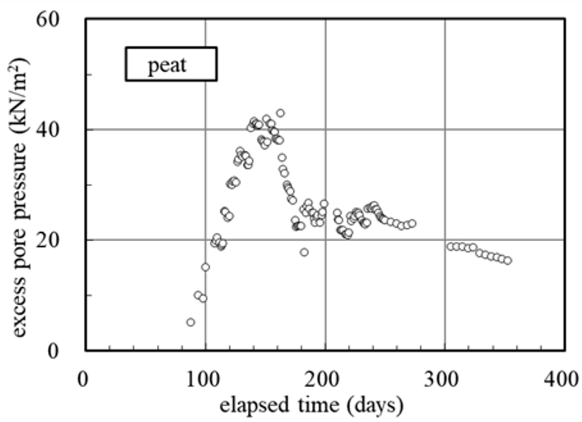

Fig. 8. Toubetsu excess pore pressure in peat vs time.

2.3 Antoniny, Poland

Two trial embankments were constructed at Antoniny, Poland. Data reported herein relates to Embankment 1 constructed without PVD to a maximum height of 3.9m in three stages ranging from 1.2 to 1.4m height using incremental lifts of 0.2m over a total period of 20 months. Soil conditions consisted of 3.1m depth of peat, 4.7m of calcareous gyttja, average organic contents of 71% & 35% respectively, overlying dense sand. An average depth to ground water level observed in peat was 0.2m. Vertical settlement profiles and horizontal deformations at the embankment toe (I2 location) for various times are presented in Figure 9. Maximum settlement of 1.87m and lateral deformation of 0.55m were observed during stage 3 hold.

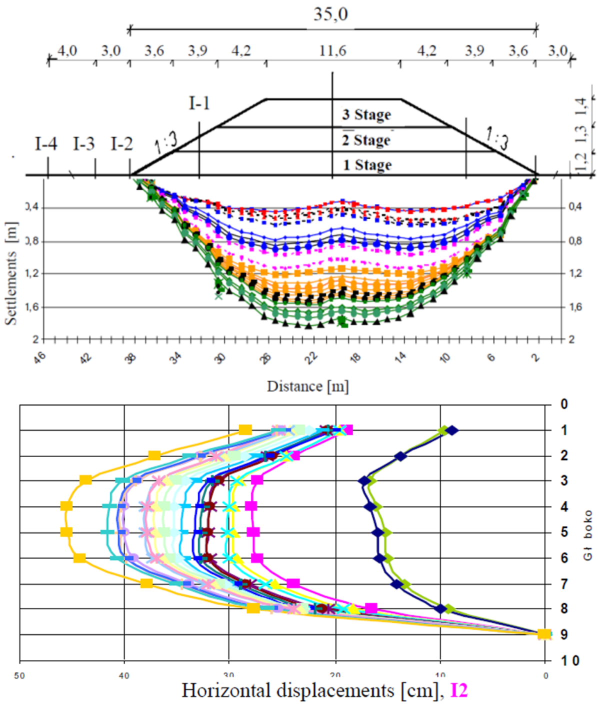

Fig. 9. Antoniny Settlement and lateral deformation profiles I2. Sas & Malinowska [8].

Loading and deformation ratio v time is presented in Figure 10. An initial maximum deformation ratio of 0.5 was observed following stage 1 loading which gradually declined to 0.3. No failure occurred but back analysis of embankment stability estimated operational factors of safety of around 1.2 following loading in stages 2 and 3 SGI Report 32 (1988).

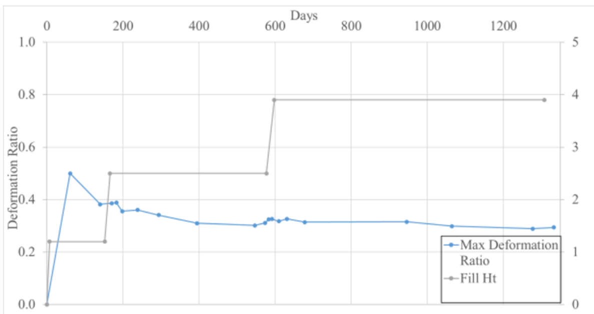

Fig. 10. Antoniny Fill Height & Deformation Ratio vs Time.

2.4 Uitdam, Netherlands

A series of full scale load tests were performed in a flat polder area on peat soils at Uitdam, Netherlands. Ground level is approximately 1.4m below the mean sea level with ground water maintained at 0.2m below ground level by pumping. Soil profile consists of a 0.3m surficial layer of marine clay over 4.5m of peat, 1m organic clay, 4m marine clay, and aeolian sand. Data presented herein relates to load test 6, which most closely resembles embankment construction, consisting of a series of 2m wide concrete slabs and water tank arranged as shown in Figure 11. While the rigid load arrangement does not fully replicate flexible embankment construction, the intensity and observation frequency of automated instrumentation in test 6 permits unique insights into peat deformation performance under staged preloading and at failure.

Loading sequence consisted of 4 preload stages applying a total of 33.6kPa over a period of 31 weeks with some surface dewatering implemented during the last 8 weeks. Final load stage 5 was rapid and achieved failure at 71.9 kPa corrected load within 31 hours. Figure 12 shows the settlement beneath the centre and maximum lateral deformations at two inclinometers 51706 and 52000 located south and north respectively of the centrally loaded slab and tank. A maximum settlement of 1.5m and maximum lateral deformation of 0.26m were observed at the end of stage 4 hold period. Figure 13 shows the deformation ratio history with an initial peak of 0.16 during stage 1 loading and a final maximum ratio of 0.17 at the end of stage 4. A factor of safety of 2.8 for conditions prior to stage 5 can be derived from the ratio of corrected load applied at the end of stage 4 and the measured failure load in stage 5. Pore water pressure measurements in the peat and organic clay layers indicate a large degree of consolidation had occurred following each of loading stages 1 – 4.

Settlement, lateral deformation and deformation ratio during stage 5 loading are shown in Figures 14 and 15, adopting a new time and deformation origin. During the initial 6 hour loading period 5 additional slabs are placed, but settlement increased by less than 20mm while lateral deformations increase by up to 30mm. This

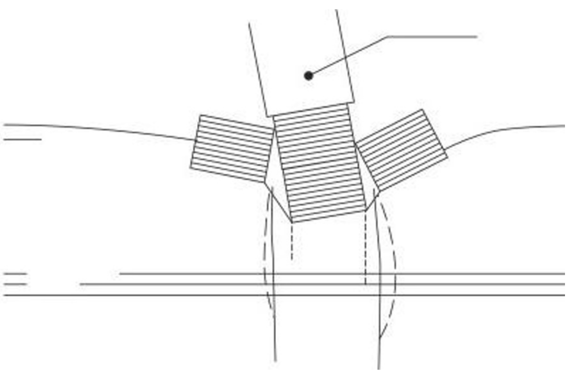

Fig. 11. Uitdam Loading arrangement Test 6. Zwanenberg & Jardine [10]

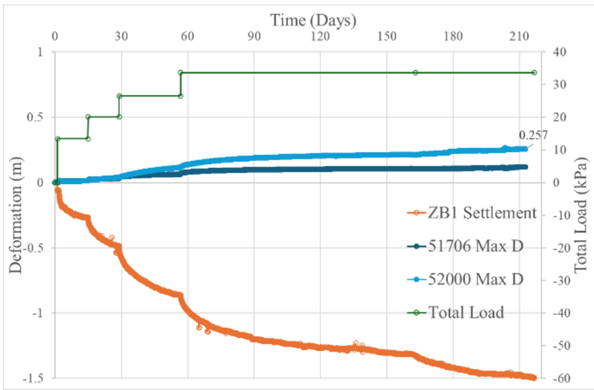

Fig. 12. Uitdam Stages 1–4 Deformations vs time.

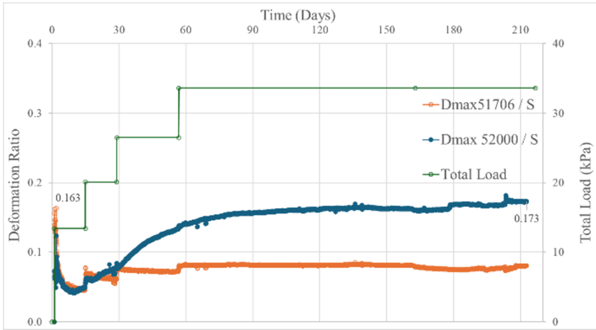

Fig. 13. Uitdam Stages 1–4 Deformation ratio vs time.

consequently produces very large deformation ratios well in excess of 1. Following placement of 5 more slabs plus the tank, loading is suspended overnight for 16 hours. During this time deformations to the northern side measured by inclinometer 52000 exceed those at the southern side at inclinometer 51706 by a factor of four, indicating that the slabs are now tilting significantly towards the north. Settlements measured at the slab centre may no longer reflect the maximum settlement at the northern slab edge. During this period deformation ratios for the northern inclinometer range from 0.86 to 0.95 and the factor of safety has reduced to 1.18. Failure occurs abruptly after a sudden reduction and then an increase in lateral deformation at inclinometer 52000.

This complex observed behaviour just prior to failure cannot easily be interpreted but the most convincing explanation relates to the following:

1. The pronounced lateral tilt of the central slabs and tank towards the north impacts the load distribution on either side of the inclinometer 52000 or directly contacts the instrument;
2. The uppermost 2m of the inclinometer 52000 has become extruded from the ground as a result of settlement and tilting of the slabs;
3. The position of maximum lateral deformation recorded by inclinometer 52000 changes significantly from near the original ground surface to depths of 3 – 4m coincident with the sudden reduction in maximum deformation.

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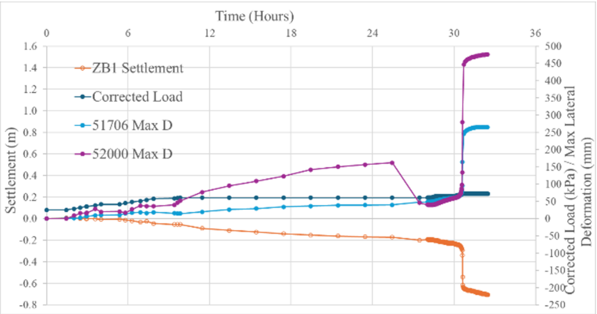

Fig. 14. Uitdam Stage 5 Deformations vs time.

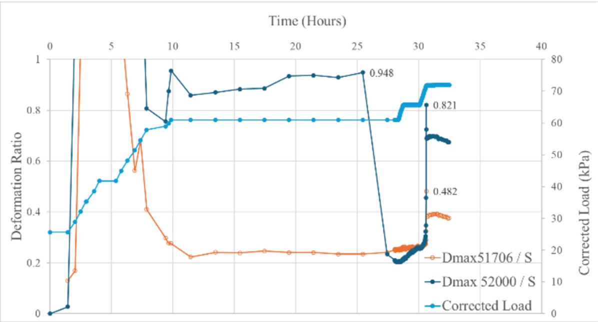

Fig. 15. Uitdam Stage 5 Deformation ratio vs time.

2.5 Glenties, Ireland

The N56 road project near Glenties, Co. Donegal, Ireland constructed embankments up to 5 m high on very soft blanket peat soils underlain by fractured granite. A 1.25 km length of road was constructed using geosynthetic basal reinforcement, multi-stage construction and 2.0 - 2.5 m high temporary surcharge, monitored by extensive instrumentation. Data from four instrumented locations referenced IC1, IC2, IC3 and IC7 are presented herein. Peat depths typically ranged from 1m to 4.8m within surcharged sections and varied significantly both longitudinally as well as transversely. A typical cross section at IC1 is shown in Figure 16.

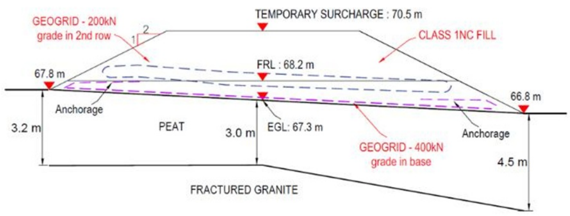

Fig. 16. Glenties Embankment cross section at IC1. Kissane et al [11]

Earthwork construction up to full surcharge height was safely achieved in 163 to 216 days, utilising 0.5m high incremental fill stages with a 2 week hold between stages, as shown in Figure 17. The final surcharge hold period ranged from 78 to 96 days. Settlements from 1.07 to 1.72m and maximum lateral deformations from 0.09 to 0.54m were observed at the end of final stage hold periods.

Graphs of normalized lateral deformation profiles versus depth and maximum deformation ratios versus time are presented in Figures 18 and 19. Note that in most locations the maximum lateral deformation is observed at depths of 1.5 to 2.5m and the deformation at the ground surface is only 40% to 60% of the maximum. Locations I1L and I2L, where the existing road was in close proximity, exhibit almost linear profiles which may result from an improved strength profile in peat due to historical compression by the nearby road pavement.

Maximum deformation ratios of 0.37, 0.35 and 0.65 occurred upon the application of first stage loading at three locations, I2R, I3L and I7R respectively. Location I7R is an outlier at a peak ratio of 0.65 which also exhibited the maximum lateral deformation of 0.54m. Peat undrained shear strengths derived from both shear wave velocity profiles and CPT ball cone profiles in the vicinity of IC7 were relatively low. Localised artesian ground water conditions were also observed at IC7. Prior to embankment construction at IC7, evidence of recent peat instability could be discerned from misalignment of relocated overhead electricity poles.

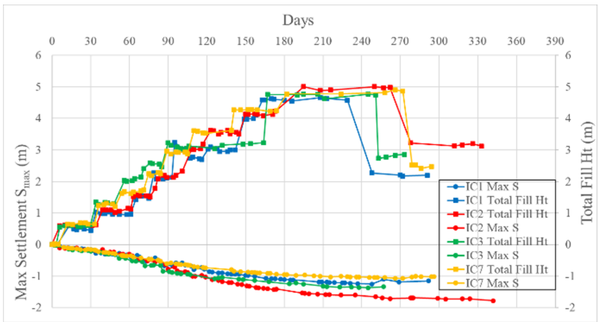

Fig. 17. Glenties Settlement and total fill height vs time.

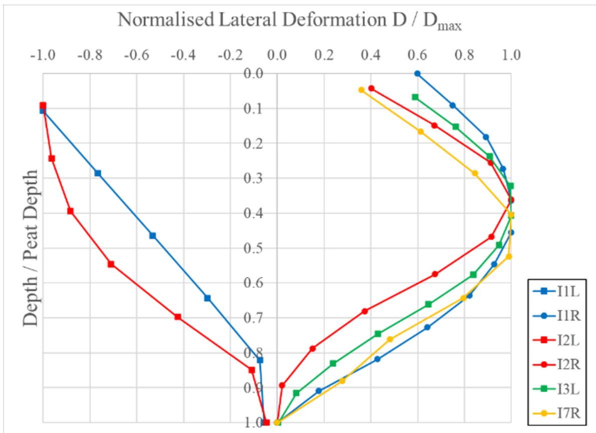

Fig. 18. Glenties Normalised Lateral Deformation Profiles.

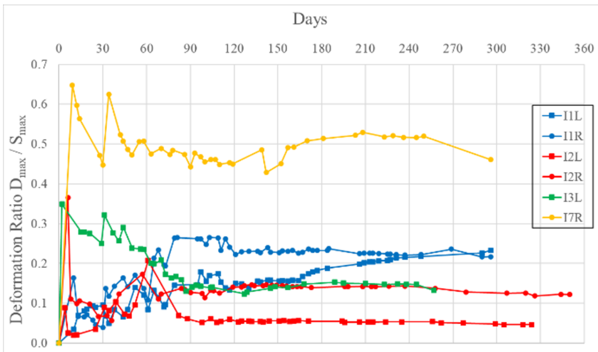

Fig. 19. Glenties Deformation Ratio vs time.

3 Discussion

3.1 Deformation and stability performance

The large range of peat characteristics, peat depths and base drainage conditions, embankment heights, slope geometry and inclusion of basal geosynthetic reinforcement are summarised on Tables 1 and 2. Durations to achieve end of construction (maximum fill height or load) range from 5 to 20 months. Drainage behaviour measured by piezometers in peat layers at the end of load stages prior to the next load stage can be broadly assessed in three categories as follows:

- fully drained (Antoniny; Uitdam stages 1-4)
- partially drained (Mukasa, Toubetsu, Glenties)
- undrained (Uitdam stage 5)

Toubetsu and Glenties are examples of stable, partially drained, multi-stage embankments constructed on moderate peat thicknesses up to 5m at filling rates of 0.1 to 0.35 m/week, broadly consistent with observations by Leroueil & Rowe [3]. Failures in peat at Tsukisappu and San Francisco both occurred when filling rates around 1 m/week or greater resulted in undrained behaviour.

Table 4 is a summary of parameters identified by Jardine [4] as significant for stability of single stage embankments. There is a very large range of maximum lateral deformations and maximum shear strains, such that no single limit value appears appropriate for embankment stability in peat. High shear strains in the range of 15% to 20% were observed for stable embankments at Toubetsu, Antoniny and Glenties IC7R as well as the failure at Uitdam stage 5. Observed maximum shear strain rates of up to 0.2% / day appears to be an indicator of stability in peat and consistent with Jardine's suggested limit for soft clay soils.

Deformation ratios also appear to follow a predictive pattern for peat soils. There is frequently a local maximum ratio observed upon initial loading (eg Antoniny, Uitdam stages 1-4 and Glenties). In other field cases there is no data available from the initial fill stage (eg Mukasa and Toubetsu). This characteristic is also noticeable in the field data published by Matsuo & Kawamura [1] Figure 1 and Buggy [5] for soft clays. Despite the complexity of behaviour and instrumentation limitations, the load test to failure at Uitdam stage 5 provides evidence that deformation ratios of circa 0.95 are predictive of imminent failure.

Figure 20 shows the relationship between maximum deformation ratio and the peat / soft layer thickness relative to the embankment / loaded area width. Studies by Matsuo & Kawamura [1] suggested a strong influence on deformation behaviour particularly during initial loading which appears to also be present in the field data. For relatively thin peat layers under wide loaded areas, vertical drainage becomes dominant. Peat typically demonstrates highly anisotropic permeability with much higher horizontal permeability.

The ranges of deformation ratios observed as local maxima during loading and at the end of construction (maximum fill height) are plotted against maximum shear strain rate in Figure 21. The author's suggested limit thresholds of 0.3 for an observational method amber "warning" and 0.6 for red "stop loading" actions are shown.

Table 4. Lateral Deformations and Shear Strains

Location	Max Lat Deform (D_{max}) (m)	Max Shear Strain (γ_{max}) (%)	Max Shear Strain Rate δγ / δt (% / day)	Fill Rate (m *kPa* / week)
Mukasa, left side	1.45	5.9	0.016	0.19 – 0.21
Toubetsu, Japan	0.63	15.8	0.20	0.35 - 0.13
Antoniny, Poland (peat / calcareous)	0.55	15.3	0.026	0.07
Uitdam, Stages 1 - 4	0.26	7.3	0.034	0.2[1] / *4*
Uitdam, Stage 5 (pre-failure)	0.16	16.8	15.9	30[1] / *600*
Glenties, IC1R	0.27	8.1	0.049	
Glenties, IC2R	0.23	8.8	0.045	0.19 –
Glenties, IC3L	0.20	5.0	0.030	0.21
Glenties, IC7R	0.54	19.7	0.108	

[1] Equivalent fill loading rates at Uitdam for $\gamma_{fill} = 20$ kN/m^3

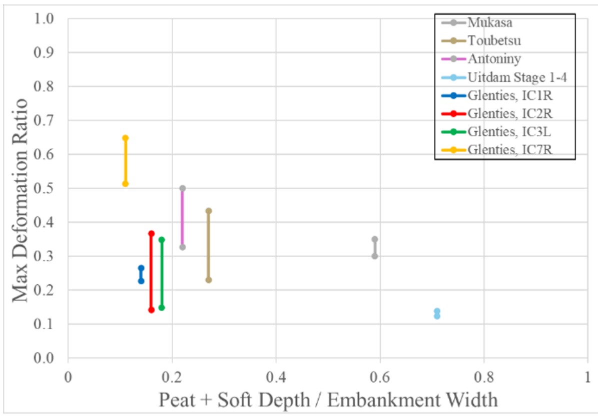

Fig. 20. Max Deformation Ratio v Peat + Soft Depth / Embankment Width.

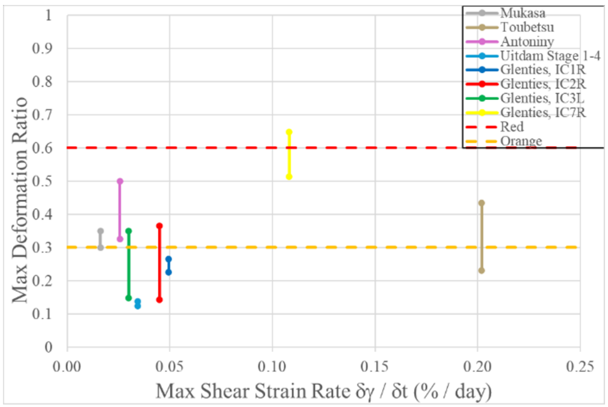

Fig. 21. Max Deformation Ratio vs Max Shear Strain Rate.

3.2 Instrumentation and data interpretation

Near surface peat deforms significantly due to natural cycles of wetting / drying, slope creep, plus impacts of nearby construction traffic and activities. At Glenties this resulted in erratic baseline inclinometer measurements of 10 to 30mm prior to embankment construction. Deformation ratios were considered unreliable until a settlement of at least 30mm was observed. A delay in observed settlements above 30mm was also noted at Uitdam upon the commencement of stage 5 loading (after an extended hold period of 160 days) with similar effects upon deformation ratios.

Piezometers are a valuable asset in any embankment instrumentation program. Measured pore pressures reflect loading, consolidation drainage and can warn of potential instability. There is a need for care in the interpretation of excess pore pressures relative to any pre-construction measurements. Baseline conditions can change due to seasonal variations; local changes in drainage or hydrogeologic conditions due to the construction of the embankment fill or drainage blanket; and large local reductions in peat permeability due to embankment loading. Piezometers embedded in the peat can settle significantly and a correction should be made, preferably based on extensometers. Deployment of multiple piezometers in a vertical profile, as implemented at Uitdam, can identify differences in behaviour between shallow and deep peat, providing insight into heterogenous permeability properties.

Inspection of the hourly instrumentation data available from Uitdam Stages 2 – 4 indicates that deformation and piezometric measurements within 24 hours of load increments are very similar, with variations being less than 1% of the peak value. The exception is Stage 1 loaded to 10 kPa, where measurements within 6 hours (ie same day as load application) are recommended.

Peak lateral deformations typically occur at depth below the peat surface and are significantly larger than those observed at the surface. The measurement of deformation profiles by means of inclinometers is therefore essential to correctly apply deformation ratio criteria in an observational method approach. Inclinometer data was obtained at the toe (I2) and from mid slope (I1) at the Antoniny site. The maximum lateral deformations were observed at mid slope during stages 2 and 3, but at the outer toe location for stage 1. This suggests that multiple inclinometer profiles may be necessary to correctly observe lateral deformations on embankment slopes of 3:1 or flatter.

4 Conclusions

Deformation performance and stability of multi-stage embankments constructed on peat soils has been assessed in detail at five field case histories plus reported failures at less well documented sites. There are a number of factors that influence the observed behaviour:

- Initial peat undrained shear strength characteristics and undrained strength ratio;
- Peat permeability, boundary drainage and consolidation characteristics;
- Embankment geometry and filling rate;
- Basal geosynthetic reinforcement.

Filling rates below 0.5m/week and incremental stage fill heights of 0.5m or less are broadly associated with stable outcomes, despite large deformations. Filling rates of near 1m/week or greater resulted in failures. Maximum shear strain rates beneath the embankment toe of up to 0.2% / day also appears to be an indicator of stability.

Use of deformation ratio to control stability in an observational approach appears valid for multi-stage embankments constructed on peat foundations. Orange "warning" and red "stop filling" limit thresholds of 0.3 and 0.6 respectively are recommended. Failure is predicted as the deformation ratio approaches or exceeds 1.0. Embankment geometry can also influence stability due to beneficial effects of horizontal drainage in peat. Higher ratios of peat thickness to embankment width tend to result in lower deformation ratios.

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References

1. M. Matsuo & K. Kawamura Diagram for construction control of embankment on soft ground Soils and Foundations **17** (3) pp 37-52 (1977)
2. F. Tavenas & S. Leroueil The behaviour of embankments on clay foundations Canadian Geot. Jrnl. **17** (2) pp 236-260 (1980)
3. S. Leroueil & Rowe Embankments over soft soil and peat. Geotechnical and Geoenvironmental Engineering Handbook. Springer, Boston. (2001) https://doi.org/10.1007/978-1-4615-1729-0_16
4. R.J. Jardine Stability and instability: soft clay embankment foundations and offshore continental slopes, Intl Symp Coastal Engng. Yokahama **2** pp99-118 (2002)
5. F. Buggy Deformation Performance and Stability Control of Multi-stage Embankments in Ireland. ICSMGE Paris (2013)
6. M. Tashiro et al Simulation of large scale deformation of ultra soft peaty ground under test embankment loading. Soils and Foundations **55**(2) pp343–358 (2015)
7. H. Hayashi & N Yamazoe Determining parameters of peat soil for finite-element analysis using the Cam Clay model. Proc. Inst. Civil Eng. – Geotech Engng. (2023) https://doi.org/10.1680/jgeen.22.00063
8. W. Sas & E. Malinowska Surcharging as a method of road embankment construction on organic soils IAEG paper 403 (2006);
9. W. Wolski et al Two stage-constructed embankments on peat SGI Report 32 (1988)
10. C. Zwanenberg & R.J.Jardine Laboratory, in situ and full-scale load tests to assess flood embankment stability on peat. Geotechnique **5** (4) pp 309-326 (2015)
11. P. Kissane et al Staged construction of surcharged embankments over peat for a national road in Co. Donegal, Ireland ECSMGE Lisbon (2024)