INTERNATIONAL SOCIETY FOR
SOIL MECHANICS AND
GEOTECHNICAL ENGINEERING

This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

https://www.issmge.org/publications/online-library

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.

The paper was published in the proceedings of the 11th International Symposium on Field Monitoring in Geomechanics and was edited by Dr. Andrew M. Ridley. The symposium was held in London, United Kingdom, 4-7 September 2022.

11TH INTERNATIONAL SYMPOSIUM FIELD MONITORING IN GEOMECHANICS

Avonmouth Severnside Enterprise Area Flood Defence Project - trial embankment construction and monitoring

Gareth MASON[1], Andrew HATHERLEY[2]

[1]Mott MacDonald Ltd, Exeter, United Kingdom
[2]Mott MacDonald Ltd, Bristol, United Kingdom
Corresponding author: Gareth Mason (gareth.mason@mottmac.com)

Abstract

Geotechnical design of embankments on soft ground requires a high level of accuracy in the determination of subsurface parameters. Unreliable parameters can lead to underestimation of settlement and potential embankment failure or overestimation of settlement and unnecessary additional fill. These projects demand a detailed understanding of the subsurface. This can be accomplished by instrumentation of a trial embankment to refine subsurface parameters predicted from ground investigation and laboratory tests.

As a case study, we present instrumentation and monitoring work for the Avonmouth Severnside Enterprise Area (ASEA) Ecology Mitigation and Flood Defence Project. It is the biggest flood defence project in the West of England's history providing 17km of flood defences along the Severn Estuary. The project is a partnership between South Gloucestershire Council, Bristol City Council and the Environment Agency.

Across most of the project, ground conditions were characterised as soft compressive Tidal Flat Deposits. The project requires the crest level of 5km of embankment to be +/-100mm of the design flood level one year after takeover by the client. Ground settlement was identified as being one of the greatest geotechnical risks to the scheme.

To investigate embankment settlement, a 4.2m high and 100m long trial embankment was constructed. This embankment was installed with monitoring instruments comprising vibrating wire piezometers, settlement plates, magnet extensometers and surface settlement monitoring points to measure the response of the underlying ground to embankment loading. Monitoring of the instruments was undertaken during construction and for 12 months afterwards.

This paper presents a summary of the investigation of the ground conditions and material properties, design of the embankment monitoring system, instrumentation installation, monitoring results and lessons learnt. The results indicate that the settlement was greater, and the rate quicker than anticipated from laboratory data, allowing the embankment construction level to be optimised, creating cost and carbon savings.

Keywords: Trial Embankment, Settlement, Soft Clays

1. Introduction

This paper describes an investigation into the settlement of a trial embankment constructed in 2020 on compressible alluvium at the site of a flood defence project.

The Avonmouth and Severnside Enterprise Area (ASEA) Ecology Mitigation and Flood Defence Project is the biggest scheme of its kind in the west of England's history. The project stretches 17km along the coast of the Severn Estuary and is a partnership between South Gloucestershire Council, Bristol City Council and the Environment Agency. The project is being designed and constructed by a BAM Nuttall Mott MacDonald joint venture (BMM JV). The flood defence work will include 5,400m of raised earth embankments, 3,050m of sheet piled walls, 1,450m of insitu reinforced concrete walls and 1,925m of precast reinforced concrete walls.

The project requires the crest of the embankments to be within +/- 100mm of the specified flood defence level one year after takeover by the client. Initial settlement sensitivity calculations indicated that there was a significant hazard of under or over building the embankment height due to uncertainty in the geotechnical parameters. Uncertainty was mainly attributed to the lack of high quality samples and hence the difficulty in measuring pre-consolidation pressure and scale effect in respect to laboratory tests that are not representative of the ground mass response to embankment loading. A major hazard to the project was the significant risk of remobilising to site to adjust embankment heights to the required elevations, if they were out of tolerance. Therefore, a trial embankment was constructed to investigate settlement and optimise the embankment construction level.

BMM JV employed Structural Soils Ltd to carry out ground investigation and instrumentation installation. The earthworks contractor (Kelston Sparkes Ltd) constructed the embankments and BAM Nuttall were the principal contractor.

2. Site location

The trial embankment site is located 1.25 km southwest of Junction 1 of the M48 and 400m south of Old Passage village, at National Grid reference ST 562 884. The site is relatively flat with a ground surface of about 7.5 metres above Ordnance Datum (mAOD). The site is bound to the west by New Passage Road, beyond which the River Severn is located flowing to the southwest. To the north, south and east, the site is bound by pastoral fields and coastal and floodplain grasslands.

3. Ground conditions and ground model

The ground model for the site was derived using geological maps by British Geological Survey, 1981, regional memoirs by British Geological Survey, 1961 and project specific ground investigations, mainly Structural Soils Ltd, 2020 and Socotec, 2021.

The trial embankment site is underlain by Holocene Tidal Flat Deposits (TFD) formed during the post-Devensian global sea-level rise. The Tidal Flat Deposits represent the Wentlooge Level Formation, and the thicknesses and descriptions are included in Table 1. Laminations of clay, silt and sand layers were not identified within the ground investigations, even though high quality samples (UT100 and Mostap) were retrieved and inspected.

Glacial Fluvial Gravels are occasionally encountered at the base of the Tidal Flat Deposits but are absent at the trial embankment site. The basement geology comprises of red mudstones of the Mercia Mudstone Group (MMG). Imported Embankment Fill (EMBF) comprising of reworked Tidal Flat Deposits was used to construct the flood embankment.

Geological unit	Description	Top of layer (mAOD)	Bottom of layer (mAOD)	Thickness (metres)
TFD Crust	Medium strength brownish grey CLAY. The clay is medium to high plasticity. This deposit represents a desiccated surface crust.	7.5	5.5	2
TFD Cohesive	Very low to low strength grey slightly sandy CLAY. Sand is fine. The clay is medium to high plasticity. Becoming low to medium strength with depth.	5.5	-7.5	13
TFD Granular	Medium dense to dense dark grey slightly gravelly slightly clayey fine to coarse SAND. Gravel is subangular to subrounded fine to coarse of sandstone and quartz.	-7.5	-19.5	12

Table 1 Ground model

4. Geotechnical design parameters

The results of the GI insitu and laboratory testing were used to determine initial consolidation settlement parameters for the cohesive TFD. The derivation of critical parameters is explained below. Due to the normally (or lightly over) consolidated nature of the TFD, a non-linear approach to calculating the primary consolidation settlement was adopted, using the Compression Index and Recompression Index values.

4.1. Void index

The void index (Iv)of the TFD was calculated from the Atterberg test data to determine whether the cohesive TFD were non-structured or structured clays. Burland, 1990 proposed a method for describing the structure level of natural clay by using the sedimentation compression line (SCL) and the intrinsic compression line (ICL). Plotting the void index against $\log\sigma_v'$ in Figure 1 shows many of the samples to be non-structured, plotting around and below the SCL. It is apparent that points falling below the ICL are associated with upper 1-2m of the soil reflecting a desiccated zone with a higher apparent over-consolidation. With increasing depth, the cohesive TFD becomes more structured, with several samples plotting above the SCL. It is generally considered that 'non-structured' clays follow the soft clay behaviours and correlations reported in literature (Burland, 1990).

4.2. Compression index

The oedometer test results were assessed for sample quality using Lunne, 2006. This established that most of the samples were of `poor` quality, although two samples were identified to be of `very good to excellent` quality. The very good to excellent quality samples were interrogated to determine the pre-consolidation pressure of the samples and the corrected Compression Index (Cc). The following parameters were derived for the TFD Cohesive material from laboratory data; Compression Index, Cc = 0.365, Recompression Index, Cr = 0.03, Pre-consolidation pressure, Pc = 85 kPa.

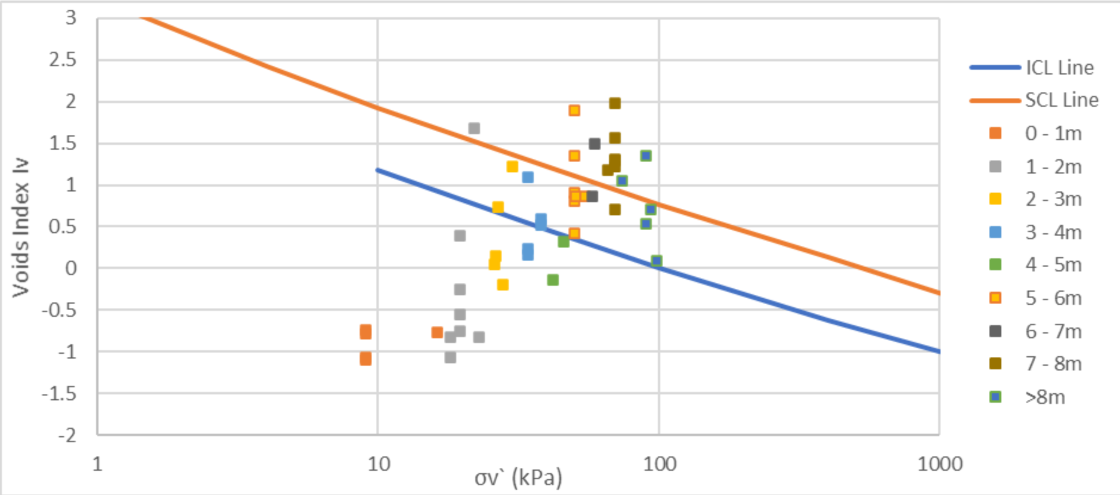


Figure 1: Void index versus effective stress and plotted with sample depth (metres)

4.3. Over consolidation ratio

Based on a pre-consolidation pressure of 85kPa calculated for the oedometer sample from a depth of 4.6m, an Over Consolidation Ratio (OCR) of 1.8 was calculated for the 'normally consolidated' TFD, assuming a unit weight of 18kN/m^3 and water level of 1m below ground level. A comparison was undertaken between the interpretated CPT undrained shear strength and normally consolidated strength profile calculated using Skempton, 1957. Based on the results, an OCR of approximately 15 was calculated for the TFD crust (reflecting the pattern in the void index discussed above) reducing to between 1.4 and 1.7 for the underlying 'normally consolidated` TFD.

4.4. Coefficient of consolidation

The vertical Coefficient of Consolidation (Cv) was derived using oedometer laboratory tests and in-situ CPT dissipation tests.

- The oedometer results suggest values typically ranging between 1 and 5 m^2/year and a peak of 22 m^2/year at the pre-consolidation pressure, reducing to between 8 and 15 m^2/year in the normally consolidated range.
- Cv derived from CPT dissipation tests, which assumed a Ch/Cv ratio of 1.25, typically ranged between 5 and 35 m^2/year.

One of the key findings of a nearby trial undertaken in 1965 (Murray, 1971) was that the observed rate of settlement was much quicker than laboratory test results suggest. The laboratory test Cv was between 3 and 9 m^2/year but trial settlement records indicated a Cv of 70 to 200 m^2/year.

5. Preliminary settlement assessment

A preliminary settlement assessment was undertaken using best estimate parameters derived from ground investigation and laboratory test data. The settlement curve is presented in Figure 3 as 'Settlement Prediction – Laboratory data'. However, uncertainty and variability of the geotechnical parameters suggested that a trial embankment would be required to increase confidence in the settlement analysis and achieve the project goals.

6. Trial embankment construction and instrumentation

The 100m long trial embankment was constructed orientated north to south to a height of 4.2m (maximum design height) with a 5m wide crest, 27m wide footprint and with 1 vertical to 3 horizontal slopes. The crest and height dimensions were selected to match the highest embankment along the flood defence. The embankment was constructed on level natural ground. The trial embankment was constructed along the line of the final flood defence and therefore could be adopted as permanent flood defence works, which avoided the requirement to remove the material following the works.

The instrumentation was grouped in four locations along the trial embankment and comprised:

- 12 no. Vibrating Wire Piezometers (VWP) installed within six boreholes with data loggers for continuous records. Six installed at 5.5m depth and six at 8.5m depth below original ground surface (prior to embankment construction) in three arrays at the embankment crest and toe. The boreholes were backfilled with a 2.5:1 (bentonite: cement) liquid grout mix.
- 2 no. magnetic extensometer arrays consisting of 7 magnets (magnet locations indicated in Figure 2) installed 2m from the landward crest. The magnetic extensometer boreholes were backfilled with a 2.5:1 (bentonite: cement) liquid grout mix.
- 2 no. settlement plates installed at original ground surface (prior to embankment construction) and approx. 2m from the landward crest.
- 25 no. surface settlement monitoring points. This consisted of two arrays (12 and 13 points) across the embankment extending up to 10m away from the toe.
- Groundwater monitoring standpipe installed within a borehole outside the embankment footprint.

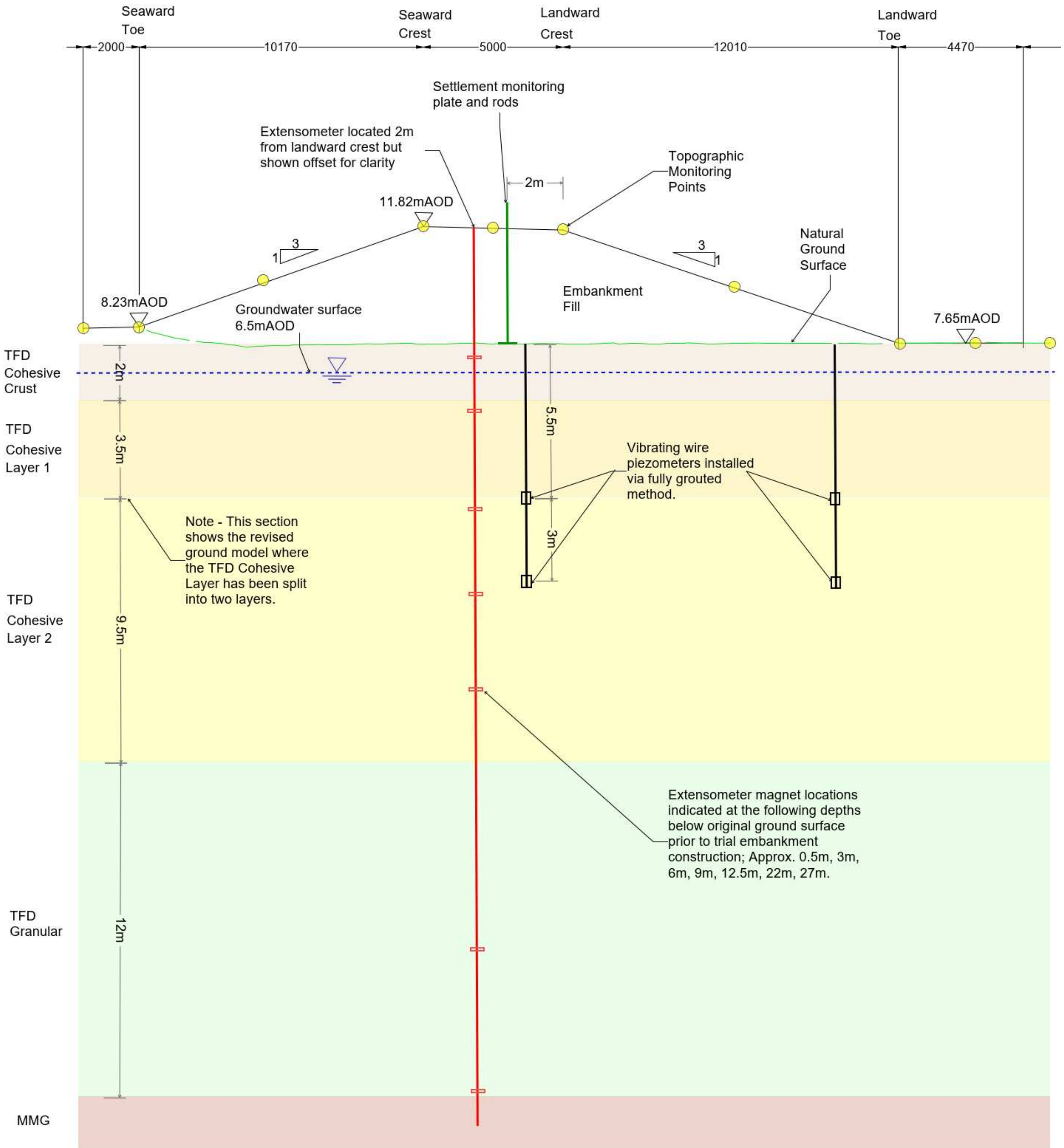


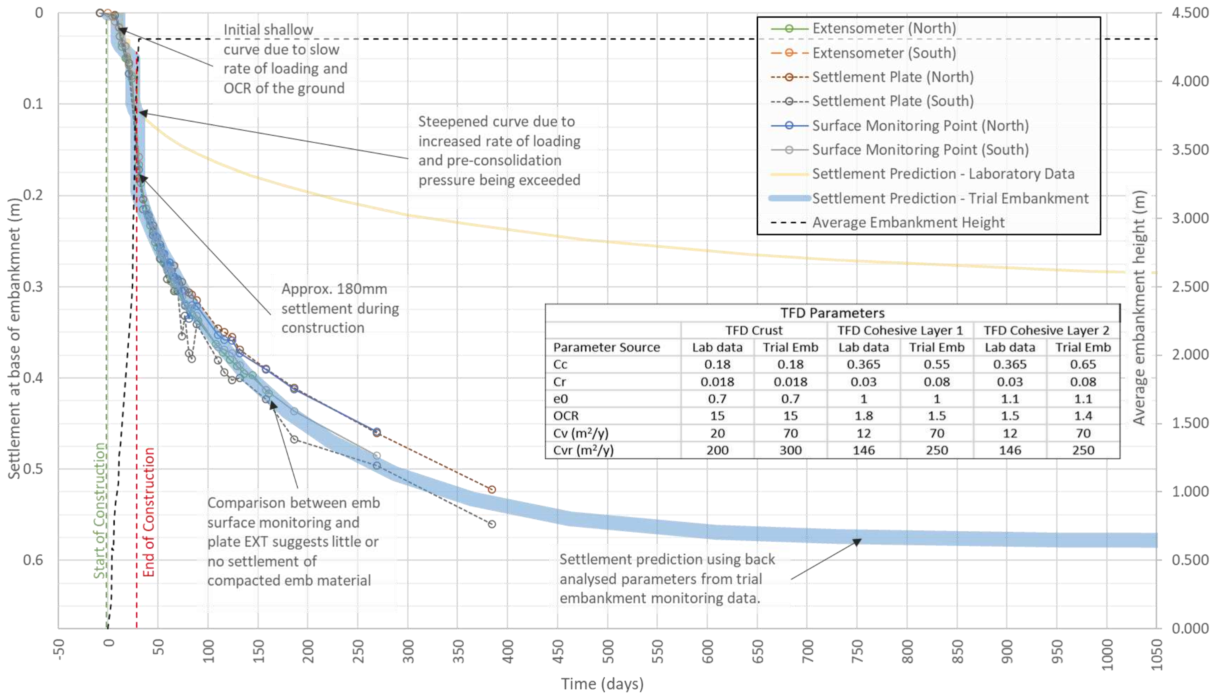
Figure 2: Cross section showing ground model and instrumentation

Regular monitoring of the instruments was undertaken during construction (several times a week). Following completion of the embankment, the monitoring frequency was gradually reduced to reflect the reduced amount of settlement.

7. Monitoring results

Trial embankment monitoring data indicates that settlement occurs quicker than anticipated based on the laboratory data (similar to Murray, 1971). The amount of settlement experienced during the trial embankment was also greater than expected.

Approximately 180mm of settlement was observed during construction with a further 240mm measured during the first 4 months of post construction monitoring (Figure 3).



TFD Parameters						
	TFD Crust		TFD Cohesive Layer 1		TFD Cohesive Layer 2	
Parameter Source	Lab data	Trial Emb	Lab data	Trial Emb	Lab data	Trial Emb
Cc	0.18	0.18	0.365	0.55	0.365	0.65
Cr	0.018	0.018	0.03	0.08	0.03	0.08
e0	0.7	0.7	1	1	1.1	1.1
OCR	15	15	1.8	1.5	1.5	1.4
Cv (m^2/y)	20	70	12	70	12	70
Cvr (m^2/y)	200	300	146	250	146	250

Figure 3: Settlement versus time

The piezometers showed excess porewater pressures at 5.5mBGL to increase from a pre-construction pressure of 44kPa to a maximum of 80kPa at the end of construction (Figure 4). During the first 4 months of post construction monitoring the pore pressures have reduced to 62kPa suggesting that at least 50% of the primary consolidation settlement has occurred.

8. Back analysis using trial embankment results

Back analysis of the trial data was undertaken to determine the consolidation parameters for the TFD. The back analysis was undertaken using the RocScience Settle3 software package using the mean 3D stress approach.

To limit the number of variables in the back analysis, only the parameters for the cohesive TFD layers were adjusted. The results from the back analysis are presented in Figure 3.

The TFD Cohesive unit was split into two layers to reflect the depths where the material generally changed from most material lying close to the ICL to more material lining at and above SCL. Additionally, using Settle3 to create a settlement curve which matches the observed settlement required the TFD Cohesive unit to be split so different parameters could be assigned to each layer.

The back analysis parameters predicted that the trial embankment will settle a total of 580mm, excluding any secondary consolidation, with most of the settlement having occurred within two years of construction (within the first year after takeover).

Sensitivity analysis was undertaken on the compression index, Cc and coefficient of consolidation, Cv parameters for the final portion of the settlement curve, to determine the likely settlement range.

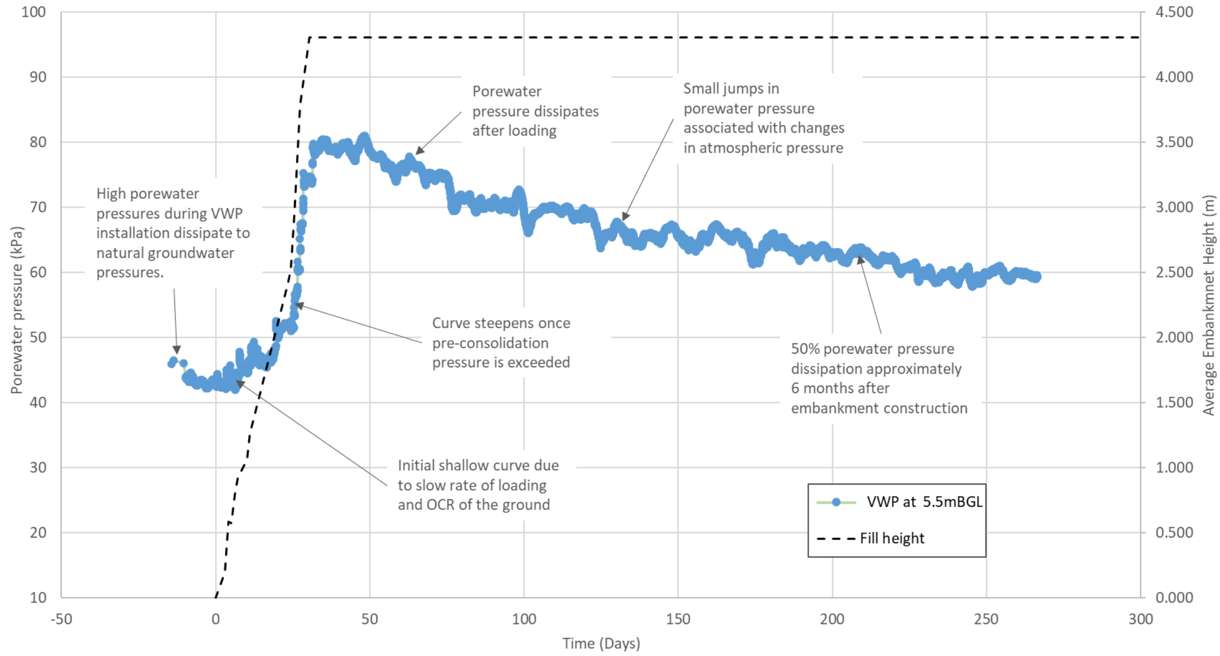


Figure 4: Porewater pressure versus time

9. Practical issues experienced during installation and monitoring

A number of lessons learnt during the course of the ground investigation, trial embankment construction and monitoring period are presented in Table 2.

Observation	**Recommendation**
Plan for a trial embankment at early GI stages.	Discuss the requirement and benefits of a trial embankment with the client at an early stage to ensure that ground investigations can be efficiently targeted.
A significant number of soil samples were assessed as poor quality according to Lunne, 2006 even though care was taken during the investigations.	Uphold investigation, sampling and transportation standards. Strive to achieve the best possible quality samples.
Incorrect pressure increments for oedometer tests can lead to inaccurate test results.	Oedometer pressure increments within lightly over-consolidated soft clays such as the TFD should be small and gradual (e.g. 10kPa increases over range of anticipated pre-consolidation pressure) to ensure that the over-consolidation details are accurately recorded.
Laboratory measurement of Cv was unreliable due to small sample size and large scale soil fabric.	Laboratory compressibility (mv) tests should be considered with field permeability values (k) to derive Cv.
Significant ground settlement around extensometers can mean that instruments become ineffective before settlement is complete.	The upper limit of settlement needs to be determined when specifying the extensometer equipment to ensure that there are sufficient lengths of telescopic sections to accommodate the anticipated settlements at specific depths.
Prompt installation of monitoring equipment is key to getting the right information at the right time.	The installation of monitoring equipment needs to be discussed early and agreed with the Contractor's programme to ensure that they understand the sequence of works, specialist personnel, duration of installation, exclusion zones, fragility of the equipment and requirements for instrument protection/signage.
The position of instrumentation needs to considered to avoid potential damage from compaction plant.	The location of instrumentation needs to be planned with the Contractor to ensure that it does not impact greatly on the construction works. Ensure data loggers are outside the limits of the embankment and haulage routes. Rod settlement plates and extensometers should not be positioned on the centreline of the crest or too close together so that compaction plant can effectively compact material.

Observation	Recommendation
The VWP requires 4 weeks to acclimatise to the ground conditions.	Allow sufficient time for VWP installation and curing time to ensure that baseline survey data can be recorded. This needs to be factored into the instrumentation and construction programme.
The position of VWPs relative to the embankment footprint needs consideration.	VWP are most effective when located beneath the centre of the trial embankment. Locating VWP at the toe of the embankment will not provide any significant benefits.
Variability in embankment construction rate.	The embankment construction rate will influence the amount of settlement that occurs during construction and therefore the embankment construction level may need to be revised to account for the Contractor's programme.

Table 2: Lessons learnt

10. Trial embankment leads to cost and carbon savings

The results of the trial embankment allowed more representative settlement parameters to be assessed giving confidence in geotechnical settlement parameters and predictions. These parameters could be used across the project with required adjustments based on location specific ground conditions. The additional confidence in the estimation of the rate and amount of settlement allowed the embankments to be constructed to a lower height, as there was greater certainty that the flood defence height could be achieved 1 year after takeover. This resulted in a 5000m3 reduction in the volume of embankment fill required to be transported to site and compacted, which is equivalent to 100,000 kg of CO_2. This is considered a minimum saving, because, without the trial embankment data, there is a risk of remobilisation of plant to modify embankments which do not achieve the specified flood defence heights.

11. Conclusions

Settlement parameters derived from back analysis of the trial embankment monitoring data indicates that settlement within 3 years of construction would be twice as much as that estimated using parameters derived from laboratory tests only. The rate of settlement calculated using trial embankment data was quicker than when laboratory data was used. These conclusions were a significant benefit to the project as the majority of the settlement occurs prior to 1 year after takeover, which allowed a review of the embankment construction level to be undertaken.

Optimisation of the embankment construction levels and increased confidence in the geotechnical parameters resulted in less imported fill and a significant cost and carbon saving. The risk of additional remedial works following demobilisation was also significantly reduced.

Acknowledgements

South Gloucestershire Council, Bristol City Council and the Environment Agency.

References

British Geological Survey (1961). Geology of the county around Monmouth and Chepstow.

British Geological Survey (1981). 1:50,000 Series (England and Wales) Sheet 250 - Chepstow.

Burland, J. B. (1990). On the compressibility and shear strength of natural clays, *Geotechnique*, 40, 329–378.

Lunne, T., Berre, T. Andersen, K.H., Strandvik, S. and Sjursen, M. (2006). Effects of sample disturbance and consolidation procedures on measured shear strength of soft marine Norwegian clays. *Canadian Geotechnical Journal*, 43, 726 - 750.

Murray, R. T. (1971). Embankments constructed on soft foundations: Settlement study at Avonmouth, *Road Research Laboratory RRL Report LR 419*.

Skempton, A. W. (1957). Discussion of the planning and design of the new Hong Kong airport. *Proceedings of the Institution of Civil Engineers*, 7, pp. 305-307.

Socotec (2021). *ASEA Factual Report on Ground Investigation*, Report No H0049-20.

Structural Soils Ltd (2020). *ASEA areas 1, 3A, 4 and 5. Factual Report on Ground Investigation*.