ENGINEERS IRELAND

RISK MANAGEMENT FRAMEWORK FOR EMBANKMENTS OVER SOFT GROUND, M17/ N17 GALWAY, IRELAND

Graham Barton, Technical Director, Jacobs, Glasgow

Paper first presented to a joint meeting of the Geotechnical Society of Ireland and Engineers Ireland West Region at National University of Ireland, Galway, on 12th November 2019

Ground Improvement at River Nanny Bridge / Tuam Bypass (Photo by Aidan Stewart)

SYNOPSIS

The N17/N18 Gort to Tuam PPP Scheme comprises 52km of mainline dual carriageway motorway, 4.2km of dual carriageway, 50No major structures and significant lengths of side road realignment. This paper describes the approach to risk management adopted for earthworks design and the monitoring of embankments constructed over soft ground. The soft ground comprised peat bog underlain by both calcareous silt and lacustrine clay. The designers were informed by technical papers in similar materials and benefitted from various phases of ground investigation. Ground improvement was employed in combination with staged construction to ensure stability with opportunity taken for surcharge to safeguard long term performance of the pavement. The thickness of fill was typically some 4 to 5m along the mainline; special measures were required for higher embankments and to protect service crossings. The predicted behaviour of soft ground was calibrated against field records to derive the operational material parameters for the soft ground.

INTRODUCTION

Soft ground was encountered beneath low-lying basins of peat in the North Section of the N17/N18 Gort to Tuam PPP Scheme.

It was determined that stage construction/ surcharge techniques could be accommodated within the construction programme for embankments in two sections of soft ground:

- 1.14km length alongside the Clare River (M17 Embankment 11).
- 1.92km length at Kilmore, south of Tuam (M17 Embankment 12/ Embankment TB1).

A piled embankment within a third section of soft ground (Embankment TB3) was subject to Contractor-led change for 'dig out and replace'.

This paper reviews the geotechnical risk management approach to decision-making, both in-design and in-construction. A selection of monitoring data is also presented, which had been used to inform the programming of the works and demonstrate that specified safety criteria were being achieved.

GEOTECHNICAL RISK MANAGEMENT

Procedures for geotechnical certification were originally brought in to regulate the reporting structure for schemes and 'to ensure geotechnical risks to projects were correctly identified and managed'; HD22/92 [Ref 1] and before that for Scotland SH4/89 [Ref 2]. The Geotechnical Risk Register was introduced later with HD22/02 [Ref 3] 'to highlight the risks and consequences of these risks together with the measures to mitigate those risks' and this was carried through to HD22/08 [Ref 4], and now to CD622 [Ref 5] and DN-ERW-03083 [Ref 6]. The definition of hazard and risk therein sets the framework for assessment, with risk being 'a measure of the likelihood of a hazard occurring and the resulting possible consequences', albeit without being prescriptive about how this is quantified. A simple assessment framework is normally adopted to allow focus to be on the thought process and give scope for engineering judgement in determining the degree of risk (see Table 1).

Table 1: Risk Assessment Framework

Scale of Likelihood	
Likelihood	Scale
Very Likely	4
Likely	3
Unlikely	2
Negligible	1

Scale of Effect	
Effect	Scale
Very High	4
High	3
Low	2
Very Low	1

Degree of Risk (Likelihood x Effect)		
13 to 16	Intolerable	Specialist Advice Required
9 to 12	Substantial	Risk Mitigation Required
5 to 8	Tolerable	Consider Risk Mitigation
1 to 4	Acceptable	None

In determining the hazard

Soft ground conditions in the Galway area, as encountered on this scheme, have been reasonably well documented in the literature. Long [Ref 7] provided an account of the troublesome 'lake bed soils' of Hanrahan [Ref 8], based on the work of various authors, and outlined the ongoing debate on the origins of the Calcareous Marls. Thick deposits of both were encountered in the soft ground areas underlying peat bog. The formation is generally attributable to groundwater-fed ephemeral lakes associated with remnant 'Turlough' features. It is conjectured that during periods of relatively static lake level, lime saturated groundwaters upwelled into these lakes and mixed with relatively acidic peat-derived waters. The resulting chemical reactions directly precipitated calcium carbonate into the interstices existing at the base of the peat and in the top of the accumulated, open-structured lacustrine clays lying beneath them. The calcium carbonate particulates were typically of silt to sand-sized gradings, with both the upper and lower boundaries of the resulting calcareous silt deposit being gradational. This mode of deposition produced calcareous silt deposits of medium to high sensitivity, with considerable loss of strength and volume occurring on disturbance due to their open-structured character.

The risk of soft ground was identified at scheme development stage and the Employer sought to provide sufficient ground investigation to allow reliable costing and design of risk mitigation measures. Eurocode guidelines [Ref 9] advise that 'knowledge of the ground conditions depends on the extent and quality of the geotechnical investigations. Such knowledge and the control of workmanship are usually more significant to fulfilling the fundamental requirements than is precision in the calculation models and partial factors'. The client-supplied ground investigation had been carried out in 4-phases over a period of 12-years and was of reasonable extent, albeit variable in quality; the aggregated scope of works for the areas of soft ground is shown in Table 2.

Table 2: Client Supplied GI.

	Embankment 11	Embankment 12/ TB1
Borehole	8	18
Trial Pit	2	11
Cone Penetrometer	12	24
Dynamic Cone	9	0
Undrained Strength	42	61
Effective Stress	3	11
Oedometer	11	23

With the peat removed, soft ground was expected to be typically 9m (Embankment 11) to 11m thick (Embankment 12/ TB1) comprising:

- Calcareous Silt. Described as very soft to soft, white-cream/beige to pale brown/yellow, slightly sandy to sandy, calcareous SILT/ CLAY with fibrous organics occurring near the upper gradational boundary with overlying peats and shell debris in the larger part.
- Lacustrine Clay. Described as very soft to firm/ compact, pale cream-brown to grey-blue mottled green/orange, slightly sandy to sandy, thinly laminated and variably organic SILT/ CLAY, with scattered shell debris and an increasing gravel content towards the base.

To safeguard both the Client (TII) and the end-users, the performance criteria for earthworks were specified as the following Construction Requirements:

- Where new construction imposes loads or stress changes on existing buildings, embankments, pavements, structures, utilities or the ground supporting embankments, pavements or structures, appropriate measures shall be taken to prevent differential settlement or damage.
- Measures shall be taken and monitoring recorded to ensure that all ground movement is prevented or is substantially complete before the execution of the road pavement.
- At any time up to Completion Date the maximum permitted change in gradient from the Design gradient shall be 0.1 percent (other items apply post construction).

In Selection of Design Parameters

In selecting design parameters for these materials, it is pertinent to further consider the Eurocode guidelines [Ref 9]. These allow for the design value for a material to be either derived from a characteristic value from data or selected directly; a characteristic value should be a cautious estimate of the mean of 'a range of values covering a large surface or volume of the ground'. The parameters adopted for analysis of stage construction were judged to be 'lower bound' and therefore represent design values rather than characteristic. The level of safety is then demonstrated by adopting Design Approach 1 Combination 2 to determine an over-design factor (equivalent to factor of safety); the objective being to make visible the attendant risk of worse case ground conditions. The guidance does state that 'less severe values than those recommended in Annex A may be used for temporary structures or transient design situations, where the likely consequences justify it'. For stage construction of the embankment, each lift could be considered to be temporary and the resulting peak excess pore water pressure is indeed transient, however, the consequences of embankment failure would be damaging to both the permanent works and the contractors programme, so the level of safety targeted was for a factor of safety >1.10 on DA1-C2. The risk management framework (Table 3) was useful in giving context for this decision.

Table 3: Parameters for Analysis of Stage Construction

Increased Cost of Ground Treatment ⇩	Increased Programme Certainty ⇨			
	Strength Parameter	Factor of Safety 1.0	Factor of Safety 1.10	Factor of Safety 1.35
	Median Value	9 (3*3)	9 (3*3)	6 (3*2)
	Moderately Conservative	6 (2*3)	6 (2*3)	4 (2*2)
	Lower Bound	6 (2*3)	4 (2*2)	1 (1*1)
	Lowest Conceivable	2 (1*2)	2 (1*2)	1 (1*1)

Acceptable (green) — Tolerable (amber) — Further Risk Mitigation required (red)

Strength testing on these materials, both in-situ and in the laboratory, had presented marked inconsistencies, which in part reflected variability of materials and in part the sensitive nature of such materials when subject to disturbance on sampling or testing. To characterize strength of the undisturbed ground, the approach of Jamiolkowski [Ref 10] was being adopted, which is predicated on a relationship between undrained strength and overburden pressure for normally consolidated clays.

A minimum value of 5 kPa was applied to both the Calcareous Silt and Lacustrine Clay, and c_u/σ_v' ratio of 0.22 assumed for in-situ conditions; a similar value being reported for very soft deposits on the Galway Eastern Approach Road [Ref 11]. The ratio is then expected to range between 0.19 and 0.30 for extension and compression conditions respectively, which broadly correlated with triaxial testing in the laboratory (see Figure 1) and was in accord with other published literature for Galway Eastern Approach [Ref 11] and Limerick Tunnel Approach Roads [Ref 12].

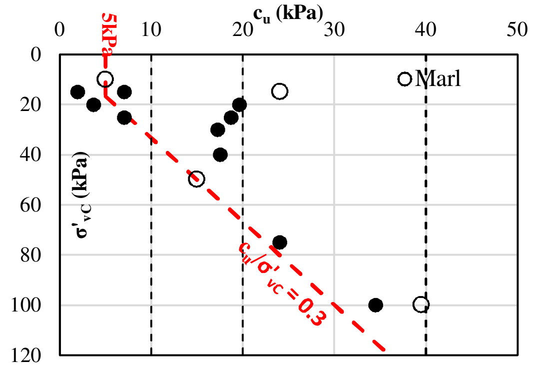

Figure 1. Consolidated undrained shear strength tests

Supplementary ground investigation was carried out to verify the parameters that had been adopted in design for the in-situ material. Cone penetrometer testing was carried out in combination with window sampling to characterize materials and this was calibrated against in-situ vane tests by varying N_{kt} for each of the two defined layers. The strength profiles derived for soft ground (see Figure 2 and Figure 3) broadly indicated that the normalized strength profile for the upper layer (Calcareous Silt) had been understated in design whilst the lower layer (Lacustrine Clay) was indeed lower bound. It follows that stability of upfill being placed in those sections relied on strength gain by consolidation and this was closely monitored by instrumentation.

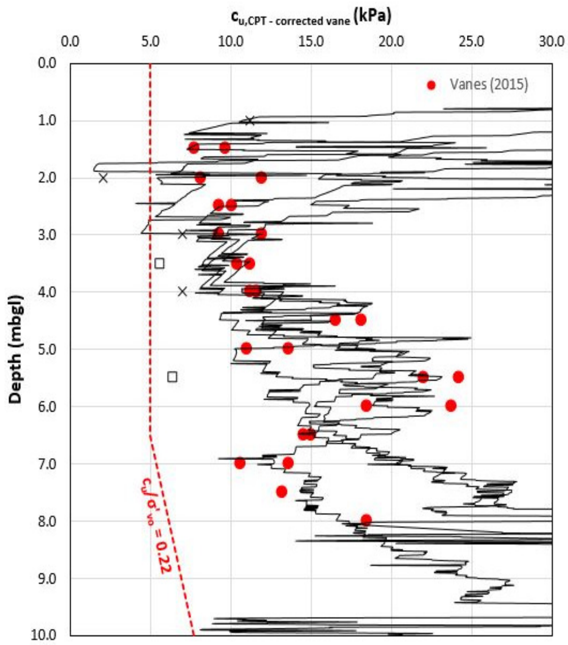

Figure 2. Calibrated CPT data (N17 Embankment 11)

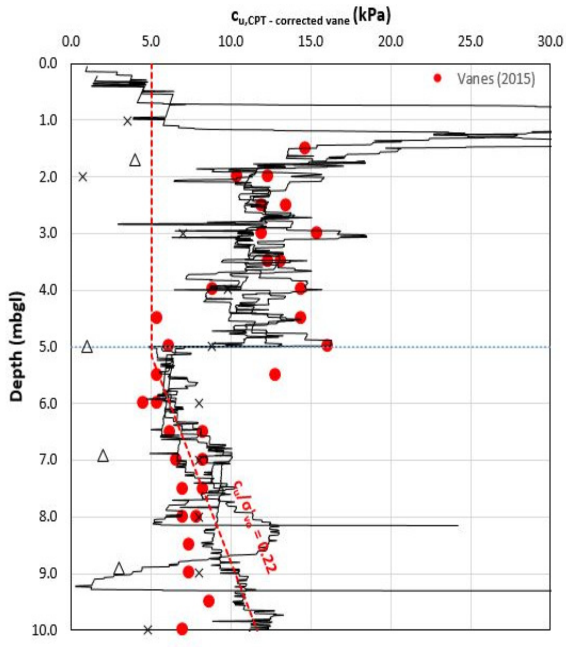

Figure 3. Calibrated CPT data (N17 Embankment 12)

The coefficient of volume compressibility (m_v) and coefficient of consolidation (c_v) derived from laboratory testing was 2.0 m^2/MN and 2.0 m^2/yr respectively for Calcareous Silt, and 1.0 m^2/MN and 1.0 m^2/yr respectively for Lacustrine Clay; Long and Rodgers [Ref 13] reported the coefficient of consolidation for calcareous silt to be somewhat higher, typically 4 to 6 m^2/year.

In Selection of Ground Improvement Option

Embankments constructed on soft ground are known to bring increased risk of instability during construction. Stage construction/ surcharge techniques are well established in earthworks practice for providing safe construction and removal of consolidation settlement to levels acceptable for pavements. The installation of band drains is to allow completion within the Contractors programme or time for surcharge to address secondary consolidation. The proposed band drain spacing was subject to Contractor change to provide greater programme certainty. Use of other improvement techniques was generally driven by programme, cost or restrictions on the land made available. The risk framework proved a useful tool for rationalising and recording decisions made for specific site situations (Table 4).

Table 4: Ground Improvement Options

	Increased Programme Requirement				
Increased Cost of Ground Treatment	Geotechnical Activity	Single Lift	Staged Construction	Staged with Surcharge	Lightweight Fill
	Starter Layer/ Drainage Layer	9 (3*3)	9 (3*3)	9 (3*3)	6 (2*3)
	Band Drains/ Stone Columns	9 (3*3)	6 (3*2)	4 (2*2)	4 (2*2)
	Basal Reinforcement/ Reinforced Earth	6 (3*2)	6 (3*2)	4 (2*2)	2 (1*2)
	Dig out Replace/ Partial or Total	4 (2*2)	2 (1*2)	2 (1*2)	2 (1*2)
	Piled Support/ LTP or Raft	1 (1*2)	1 (1*2)	1 (1*1)	1 (1*1)

Acceptable   Tolerable   Further Risk Mitigation required

Two embankment sections could be accommodated within the construction programme comprising:

- M17 Embankment 11 ch 20+640 to 21+780 (1.14 km length)
- M17 Embankment 12/ Embankment TB1 ch 24+100 to 25+420 and TB 0+00 to 0+600 (1.92 km)

Other special measures implemented within these sections were to safeguard an existing water main (floating slab), achieve early upfill of abutments for piling of an overbridge (reinforced earth), and the existing N17 where widening required replacement of peat whilst maintaining live traffic through the works (lightweight fill).

A third area of soft ground (Embankment TB3), however, was up to 11m high and considered to be not suitable for stage construction. This section incorporated a bridge crossing of the river Nanny and a railway bridge. A pile-supported Load Transfer Platform (LTP) was proposed with dig out and replacement on both approaches with the extent of dig out being curtailed due to unresolved design risk relating to:

- Insufficient land available to contain buried slopes
- Instability of excavated face in soft ground
- Disposal of a large volume of soft ground
- Excavation within land liable to flooding

Design effort was made to reduce the height of embankment and in construction there was a Contractor-led change to extend 'dig out and replace' through the full area; thereby removing the pile supported LTP. Driving this change was a confidence gained in the stand-up time of soft deposits, or at least a methodology developed to allow safe excavation thereof; availability of sufficient rock fill to allow steepening of the buried slope; and a method devised for supporting the River Nanny during excavation.

Analysis and Monitoring

Analyses for determining construction stages considered both the SHANSEP [Ref 14] approach to improvement of undrained strength (Stress History and Normalized Soil Engineering Properties) and the effective stress approach to analysis of the undrained condition. Commercially available software (Settle 3-D and SLOPE/W) readily allowed an iterative methodology to be developed for the specification of each load cycle based on a predicted porewater pressure response to load, consolidation settlement and resulting strength improvement (see Figure 4). The design strategy being that field performance is then to be monitored with vibrating wire piezometers, inclinometers, extensometers and settlement monitoring plates.

Notable issues arising from analysis include;

- Critical slip circles for the modified-cu analysis differ markedly from those for c-phi analyses and require zoning of layers beneath the shoulders of embankment.
- Surcharge allowance for plant and equipment has a limiting effect for the deeper critical slip circles associated with the modified cu approach. A 10 kPa surcharge for construction plant equated to a lift of 500 mm and being a transient load doesn't bring a

strength benefit from consolidation. More exact calculations were necessary for specific plant to better model surcharge.

- Compensation filling of up to 250 mm may be required for a given stage to regulate ongoing settlement, and there is benefit in placing this well in advance of the next stage of upfill.
- Design based on lower bound parameters requires acceptance of a low factor of safety, on the basis that excess porewater pressure is a transient condition.

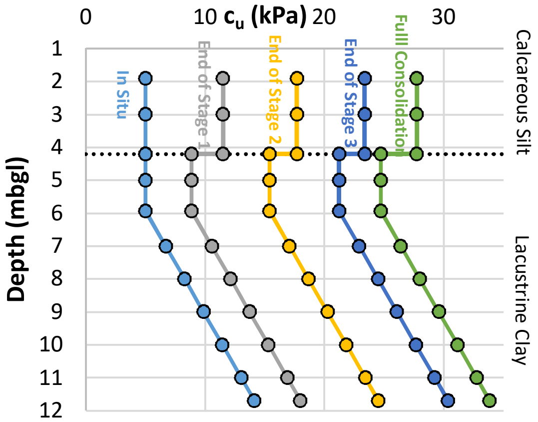

Figure 4. Typical modified-Cu for each stage

In broad terms, the allowable initial lift for the various areas was some 1.8 m to 2.3 m of fill. The scheduled hold period of 3-months was to achieve 50 to 75% dissipation of excess pore water pressure depending on the relative proportion of calcareous silt and lacustrine clay. In practice this incorporated an initial ‘working platform’ placed to allow installation of band drains and other monitoring instrumentation. Thereafter, upfill was limited to 600 mm lifts, with 800 mm lifts where stabilized with side slope berms, and 1,000 mm lifts where stabilized with basal reinforcement; each with a scheduled 3 to 4-month hold period. It follows that a 12-month period was required to upfill a 5.0 m high embankment.

Vibrating wire piezometers proved responsive to the placing of fill (see Figure 6 to Figure 9). The installations required careful detailing of the band drain layout to obtain reliable readings. The continuous-read provided good context for understanding fluctuations in pressure, which included movement of plant and flooding of the site. Fluctuations coinciding with changing levels in the River Nanny were never fully explained, whilst pump activity by the Contractor at Kilmore Junction (Embankment 12/ TB1) was extended when it proved to be beneficial to consolidation. In general, other than issues with lack of a reliable baseline, the monitoring equipment proved sufficiently robust to successfully provide continuous records over a 24-month period of construction activity.

In broad terms, the excess porewater pressure response to load was as expected; one location proved atypical and required modification to the fill schedule to accommodate abnormally high porewater pressure. Dissipation of excess porewater pressure typically achieved the scheduled 50% to 75% over 3-months, with later stages tending to dissipate at increased rates, which is another feature reported in the literature, being attributed to the establishment of preferential drainage pathways.

The progress of consolidation settlement was closely monitored (see Figure 10 to Figure 13, and the performance of the ground broadly matched the predicted behaviour:

- 1190 mm settlement recorded at N17 Embankment 11 (ch21+400). The predicted consolidation settlement with surcharge had been up to 1200 mm.
- 1500 mm settlement recorded at N17 Embankment 12 (ch 25+400). The predicted consolidation settlement with surcharge had been up to 1550 mm.

Conclusion

Different ground improvement techniques were proposed for specific situations in the soft ground areas, each being subject to risk review. A proposed band drain spacing was modified by the Contractor to provide greater programme certainty, and a piled LTP solution replaced with dig out to reduce long term risk.

The ground investigation supplied at tender stage proved to be sufficient for costing the works, however, further specialist insitu testing was required to provide sufficient level of confidence that the parameters selected for design would be suitably robust. This was subject to risk review, as was progressing with safety factors for stage construction less severe than what would normally be adopted for design of permanent works.

Monitoring of the soft ground response to imposed load was an important mitigation factor in managing the risk of embankment instability during stage construction. The monitoring carried out by the Contractor broadly verified the predicted behaviour, indeed allowing some sections to progress ahead of schedule.

There was opportunity for operational material parameters for the soft ground (Calcareous Silt and Lacustrine Clay) to be back-analysed to calibrate fill proposals ahead of the third lift. The following parameters were derived:

- coefficient of volume compressibility (m_v) = 2.3 m^2/MN (range 1.5 to 3.0 m^2/MN)
- coefficient of consolidation (c_v) = 2.0 m^2/yr (range 1.0 to 5.0 m^2/yr)

It is accepted that within this figure there will be different behaviours from different layers, however, this contributed to giving comfort on programme delivery. In the event, the later stages tended to dissipate slightly quickly and settle somewhat less than the earlier filling. This allowed surcharge to be placed and held to benefit the performance of the pavement.

The monitoring has confirmed that the Construction Requirement that ‘all ground movement is prevented or is substantially complete before the execution of the road pavement’ had generally been met. Here-on, secondary compression is expected to be ongoing over a very long period. Surcharge was designed to mitigate the magnitude of future secondary compression and thereby remain within the Construction Requirement that ‘the maximum permitted change in gradient from the Design gradient shall be 0.1 percent.’

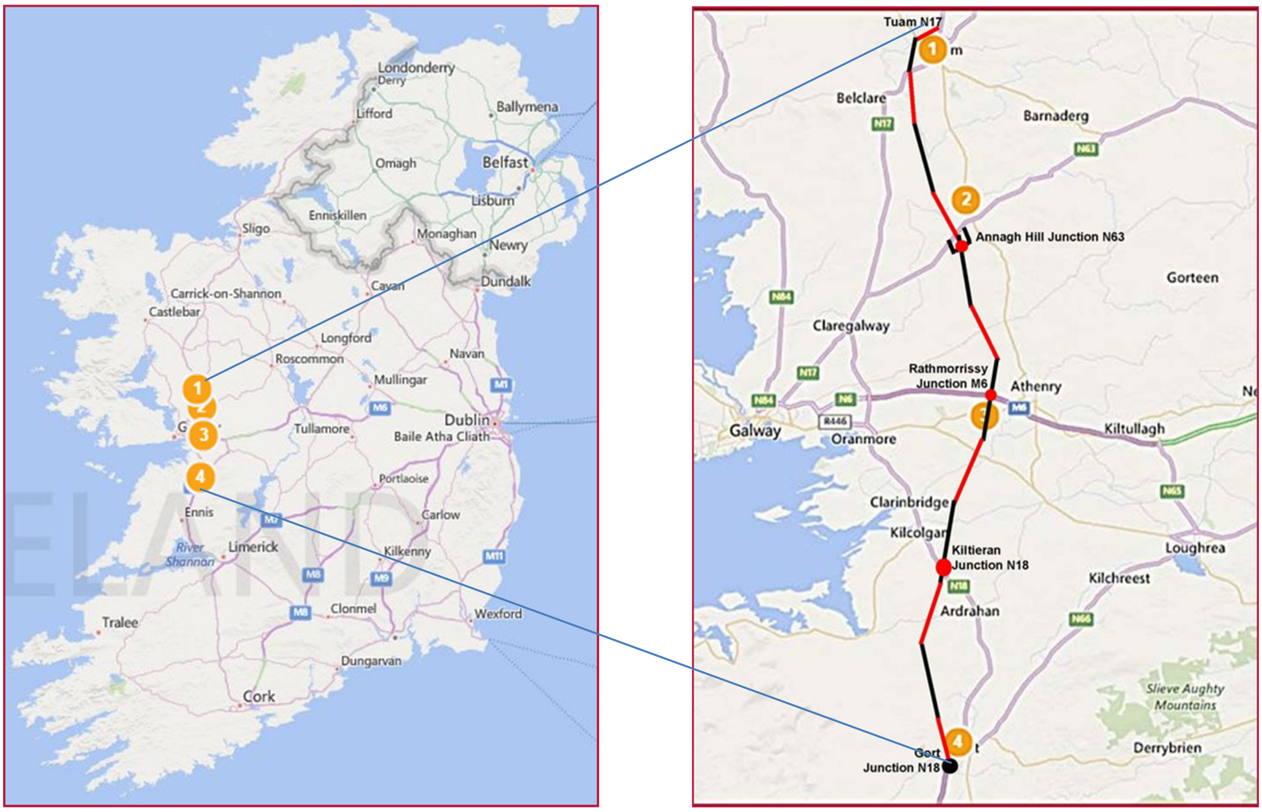

Figure 5 Location N17/ N18 Gort to Tuam Scheme

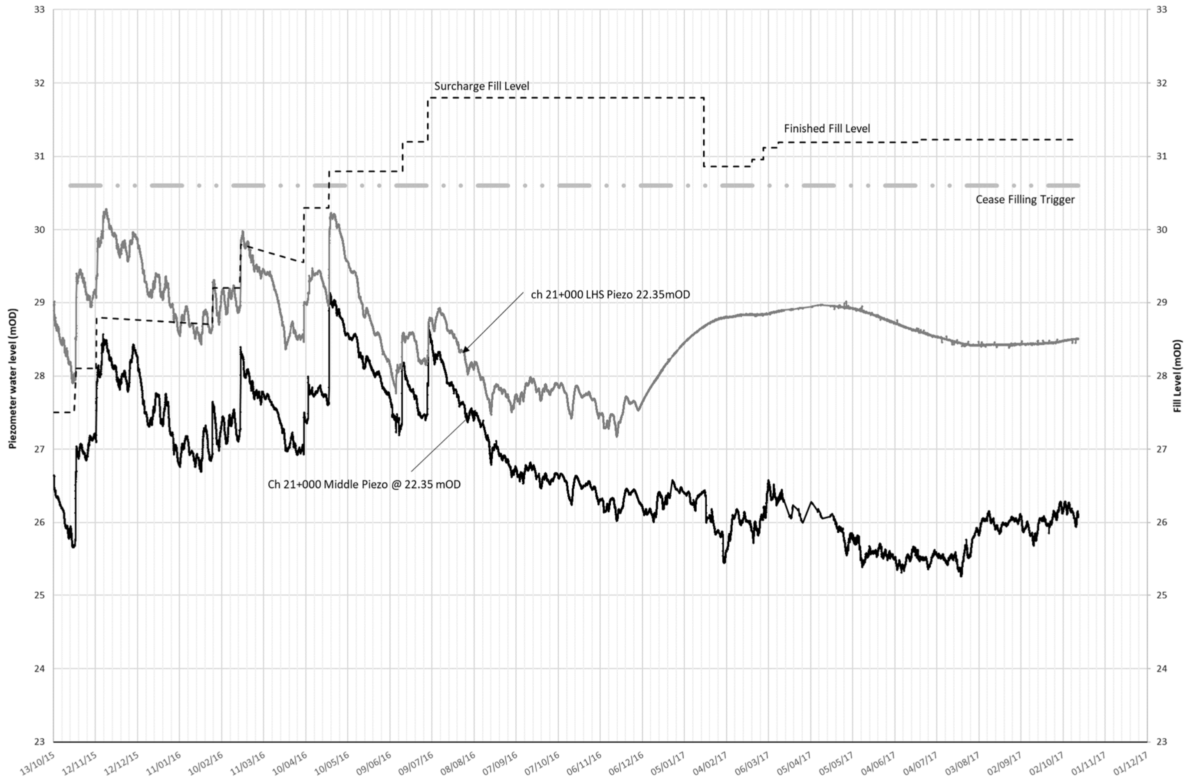

Figure 6. Piezometer water level with time - Embankment 11 (Ch 21+00).

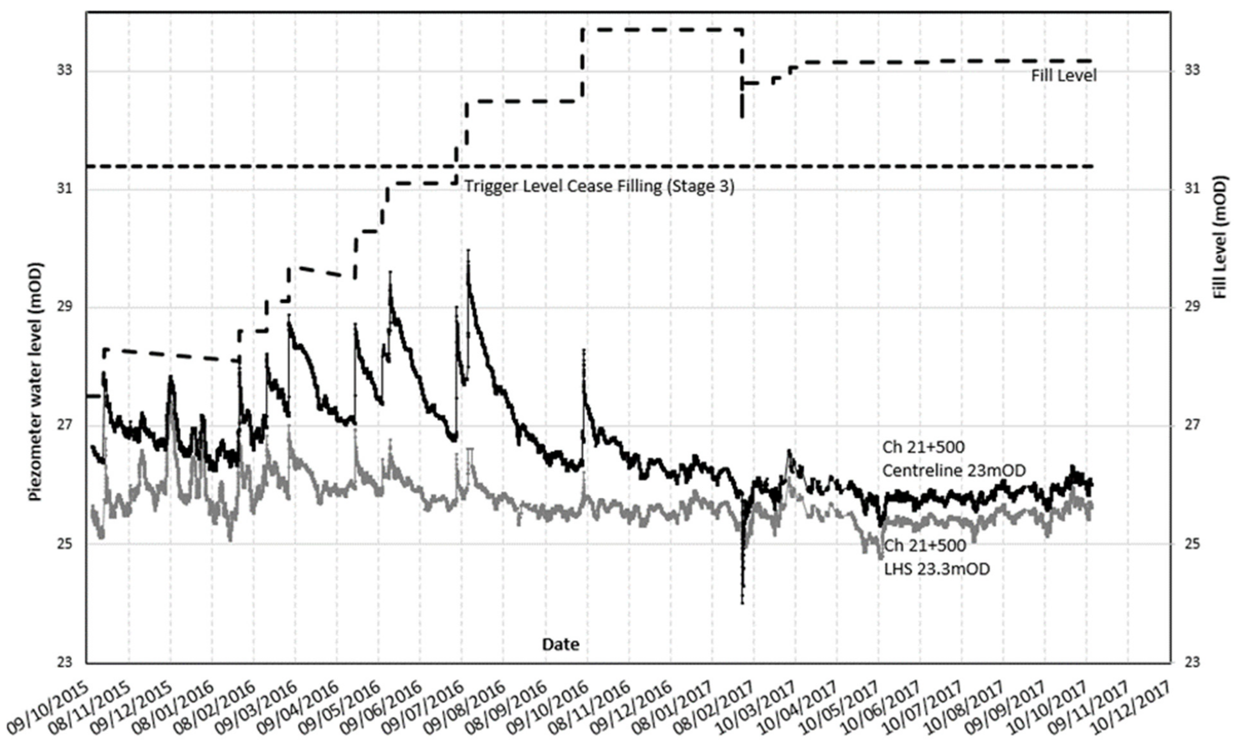

Figure 7. Piezometer water level with time - Embankment 11 (Ch 21+500).

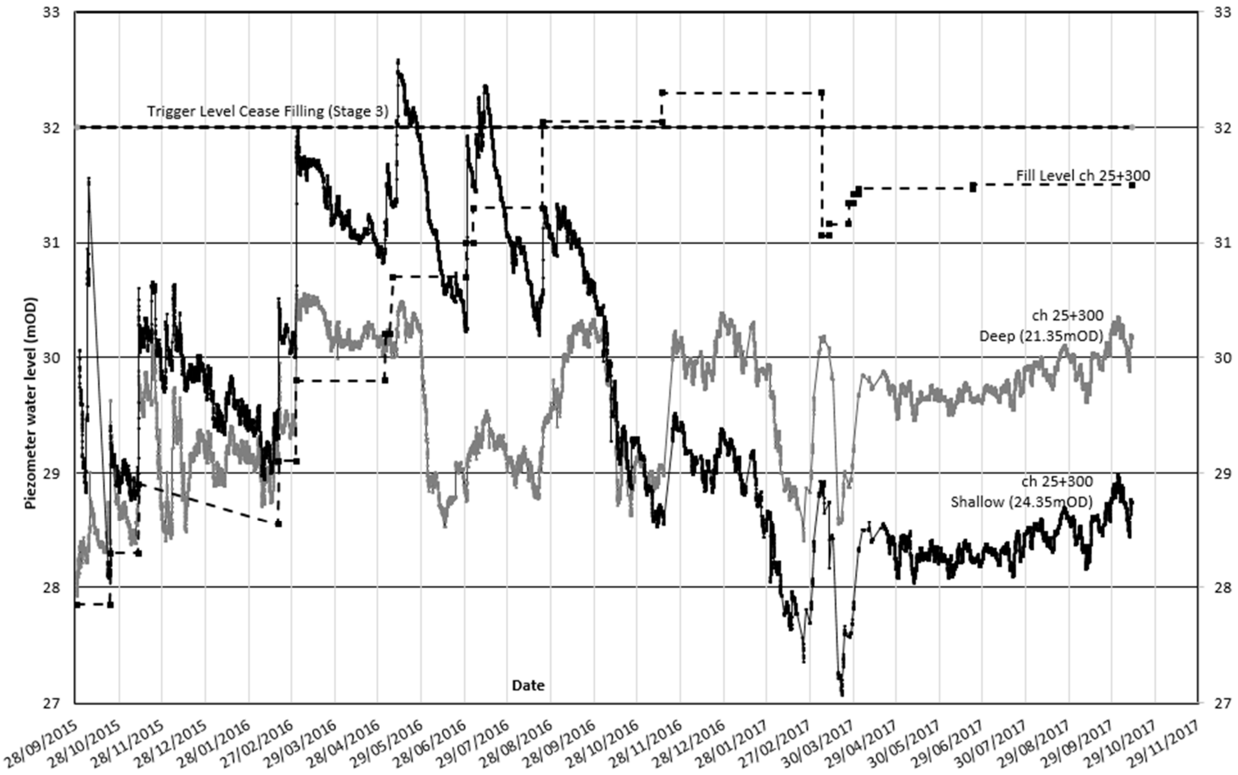

Figure 8. Piezometer water level with time - Embankment 12 (Ch 25+300).

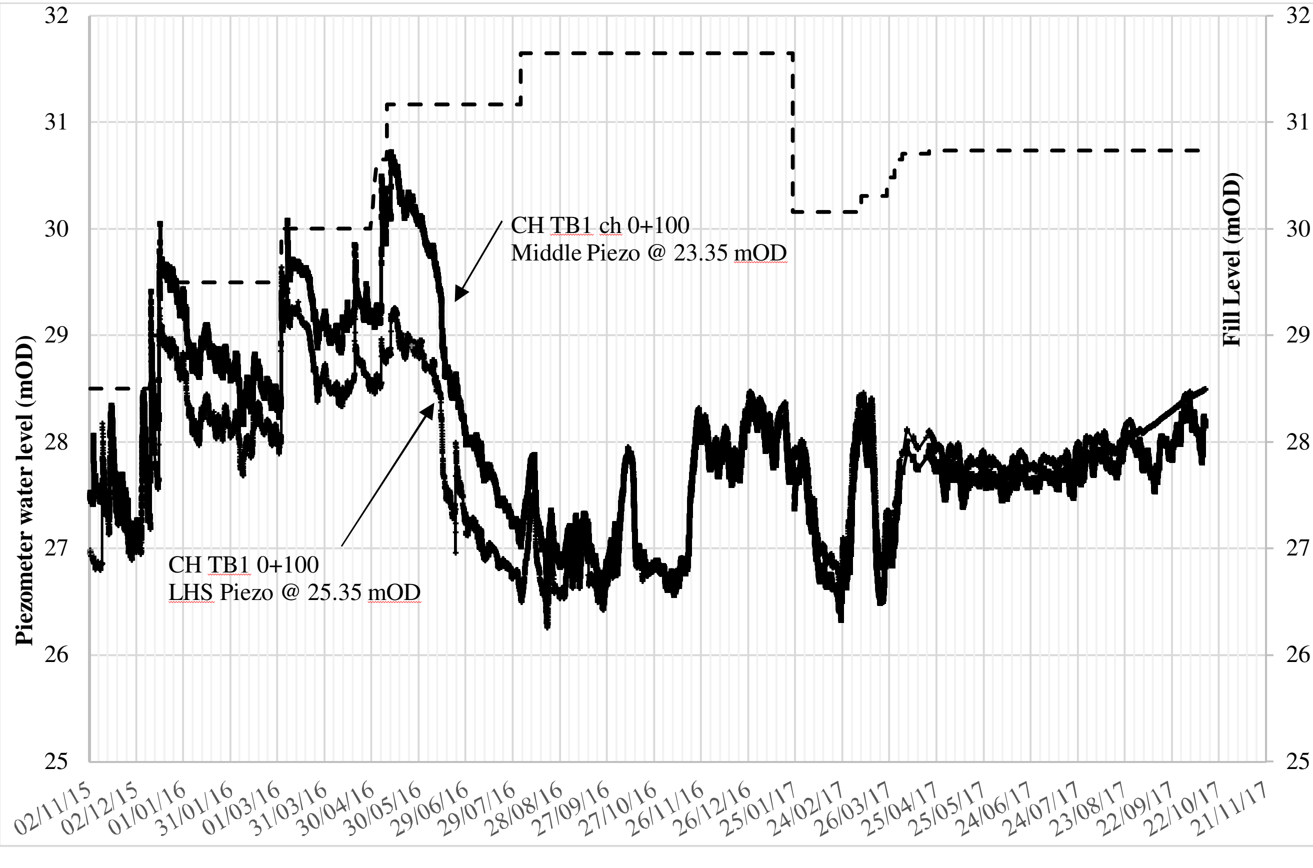

Figure 9. Piezometer water level with time - Embankment TB1 (Ch 0+150).

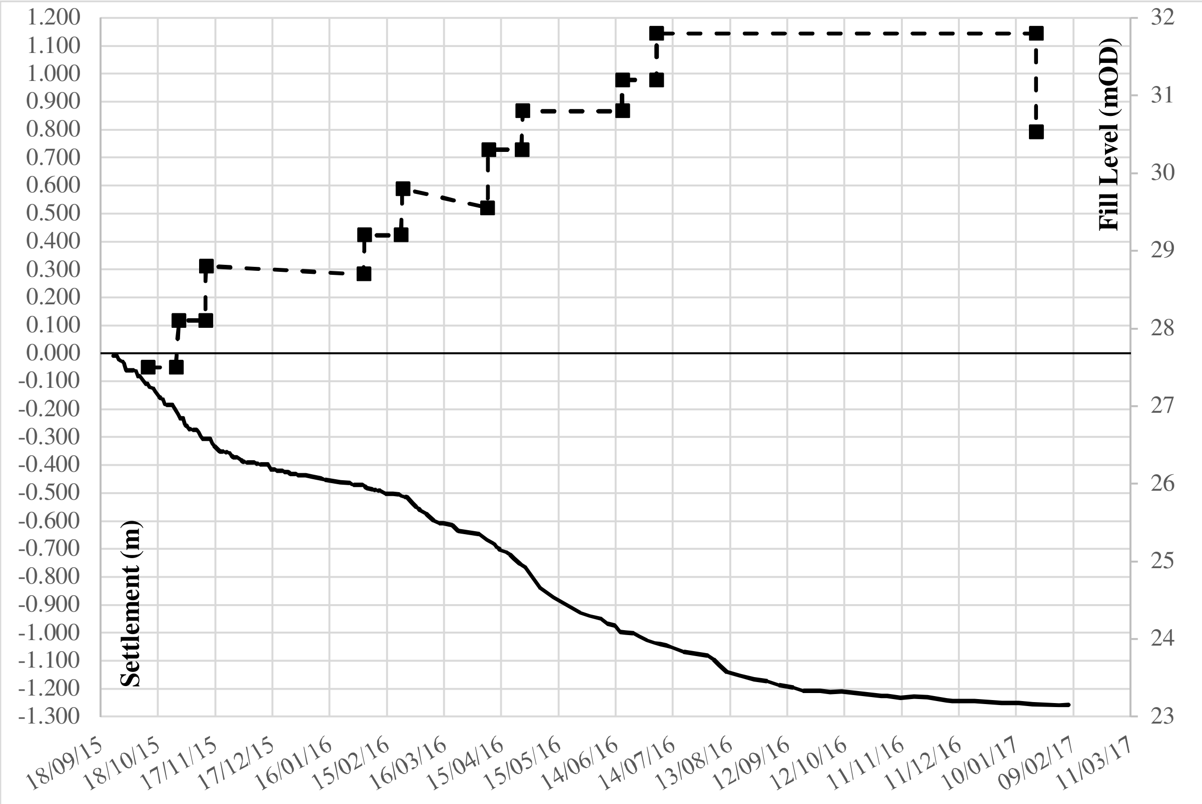

Figure 10. Settlement with time - Embankment 11 (Ch 21+000).

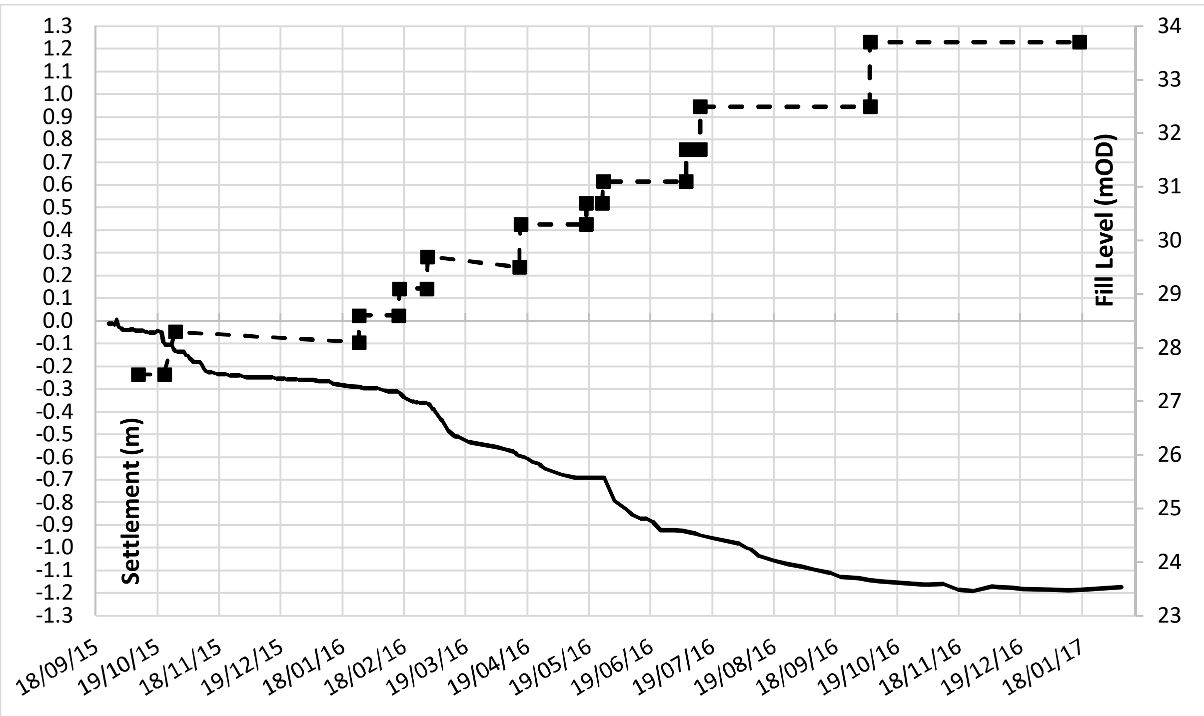

Figure 11. Settlement with time - Embankment 11 (Ch 21+400).

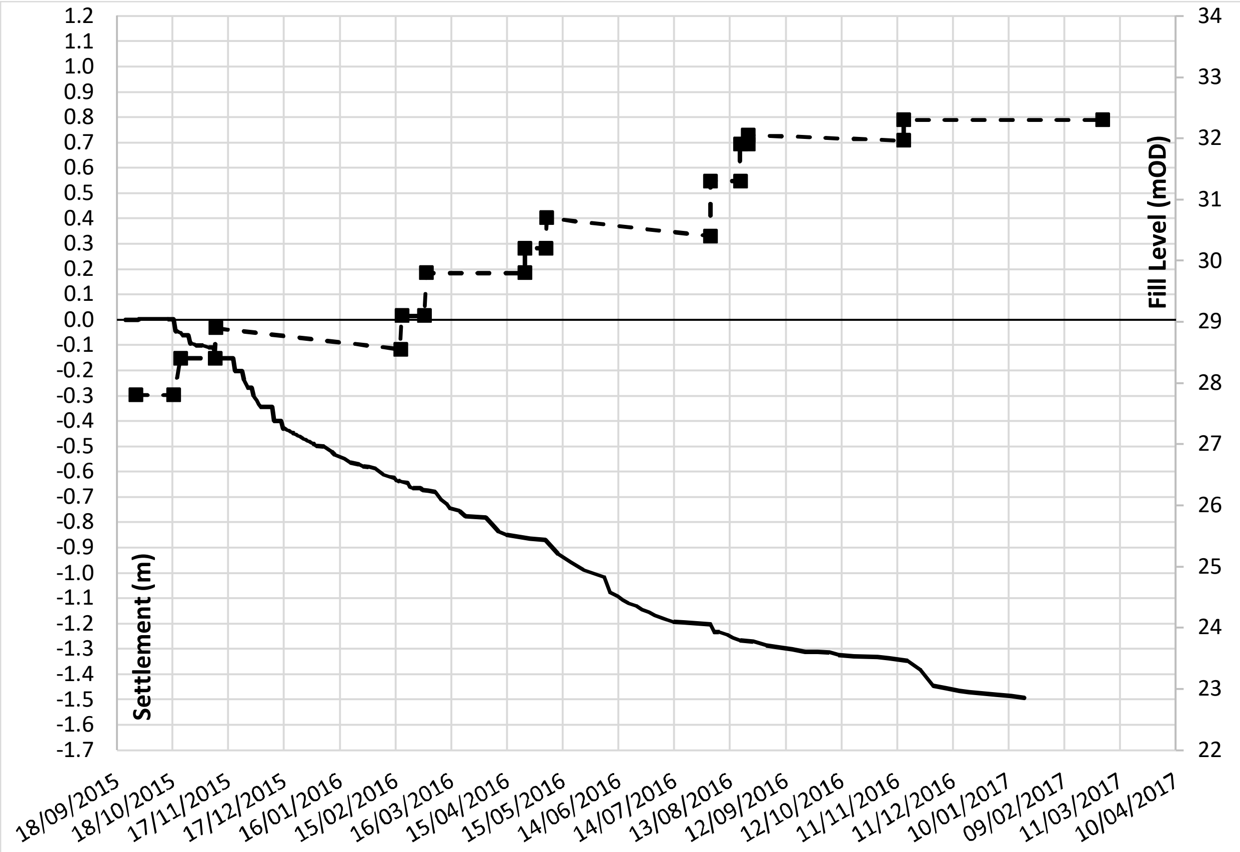

Figure 12. Settlement with time - Embankment 12 (Ch 25+400).

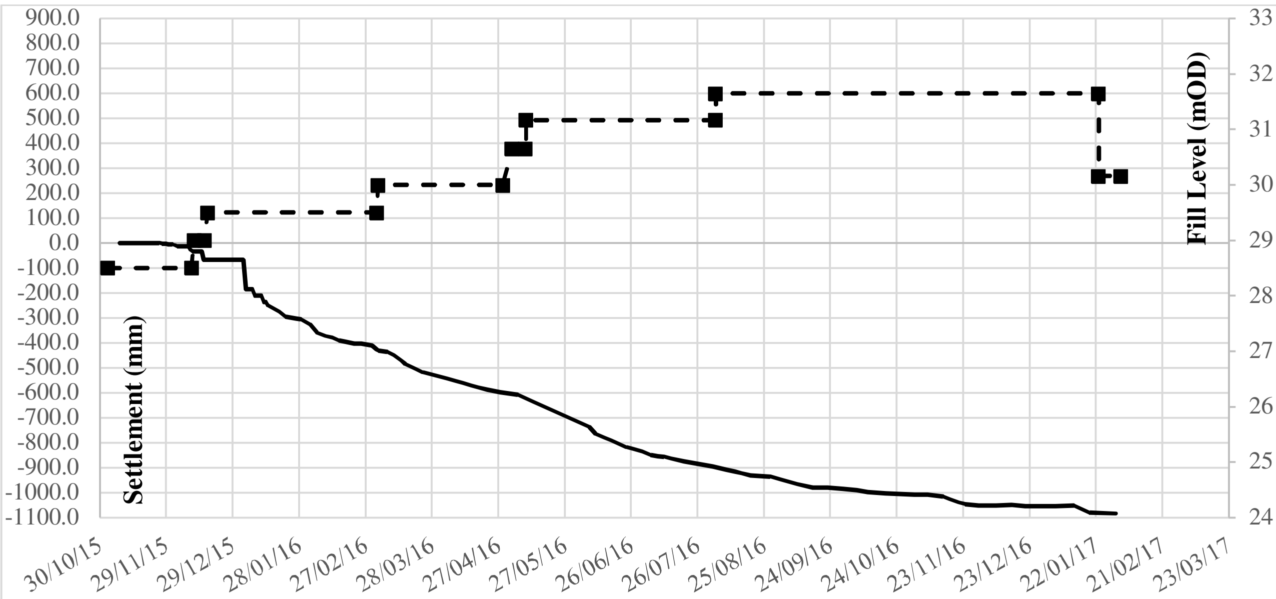

Figure 13. Settlement with time - Embankment TB1 (Ch 00+150).

REFERENCES

[1] HD22/92 Geotechnical Certification (1992). Design Manual for Roads and Bridges. *Published by The Department of Transport.*

[2] SH4/89 Geotechnical Certification Procedures (1989). *Published by Roads Directorate, Scottish Development Department.*

[3] HD22/02 Managing Geotechnical Risk (2002). Design Manual for Roads and Bridges. *Published by The Highways Agency.*

[4] HD22/08 Managing Geotechnical Risk (2008). Design Manual for Roads and Bridges. *Published by The Highways Agency.*

[5] CD 622 Managing Geotechnical Risk (2019). Design Manual for Roads and Bridges. *Published by Highways England.*

[6] DN-ERW-03083 Managing Geotechnical Risk (2019). *Published by Transport Infrastructure Ireland.*

[7] Long (2018). Geotechnical Properties of Irish Compressible Soils. *2nd E.T.Hanrahan Memorial Lecture, Geotechnical Society of Ireland.*

[8] Hanrahan (1979). Irish Compressible Soils: Origin and Characteristics. *An Foras Forbartha.*

[9] IS EN 1997-1:2005+AC:2009. Eurocode 7: Geotechnical design – Part 1: General rules (Including Irish National Annex 2007).

[10] Jamiolkowski M., Ladd C., Germaine J., and Lancellotta R. (1985), 'New developments in field and laboratory testing of soils', *Proceedings of 11th International Conference on Soil Mechanics and Foundation Engineering, San Francisco*. Vol 1, 57-153.

[11] Flood & Eising (1987). The use of Vertical Band Drains in the Construction of the Galway Eastern Approach Road. *Proceedings of 9th European Conference on Soil Mechanics, Dublin*. Vol 1, pp 417 – 529.

[12] Buggy & Curran (2011). Limerick Tunnel Approach Roads – Design, Construction and Performance. *Engineers Ireland, Geotechnical Society of Ireland.*

[13] Long M., and Rodgers M. (1995). Geotechnical behaviour of very soft calcareous soils in Ireland. *Proceedings of XI European Conference on Soil Mech Copenhagen,* 8-103-8-108.

[14] Ladd C., and Foott R (1974). New design procedures for stability of soft clays. *Journal Geotechnical Division of American Society of Civil Engineers,* GT7, 763-786.

ACKNOWLEDGEMENT

The author gratefully acknowledges the support given by many colleagues from CH2M and Halcrow Barry during design and construction of the works; in particular the site team, Colin Osborne & Brian Hastings who diligently collected and collated the monitoring data over a period of 2-years. The commitment from the site team of Roadbridge Ltd to safeguard instrumentation and integrate this with programming their management of earthworks is much appreciated.