LIMERICK TUNNEL APPROACH ROADS - DESIGN, CONSTRUCTION AND PERFORMANCE

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Cover Photo: Aerial View of Mainline earthworks North of Shannon. Bridge B09 Meelick Creek & Toll Plaza in foreground, Coonagh West Interchange, Casting Basin and Dredge Disposal Ponds adjacent to River Shannon in background.

SYNOPSIS

Limerick Tunnel PPP required 10km of dual carriageway plus two toll plazas to be constructed predominantly on embankments typically 3 to 8 m high on deep soft alluvium soils in the tunnel approaches. The soft alluvium comprises mainly organic silt / clay to depths of up to 13 m, being underlain by deposits of glacial tills and/or limestone. The embankments employed a range of geotechnical solutions from full or partial excavation and replacement of soft alluvium soils to surcharged, multi-stage construction using prefabricated vertical drains and basal geosynthetic reinforcement. Temporary surcharge fill heights and hold durations were designed to reduce long term creep settlement and the embankments were fully instrumented to monitor both stability and settlement performance. Construction commenced in June 2006 and was successfully completed in July 2010. Earthworks required the importation of over 3 million m³ of fill and careful sequencing of temporary surcharge fill materials to achieve an efficient reuse of fill materials. A short 600m section of the road was constructed on a rock fill causeway built over Bunlucky Lake. Data on the performance of pore pressures, horizontal deformations and vertical settlements plus basal reinforcement strains are included in the paper.
INTRODUCTION

PROJECT DESCRIPTION

The Limerick Tunnel PPP project is located to the south and west of Limerick City and provides a dual carriageway bypass of the city for traffic on the N18 from Clare and Shannon Airport connecting to the existing M7 bypass at Rossbrien Interchange. Interchanges are also provided at the N69 Dock Road and at Coonagh West to a new 1.3 km long single carriageway link road to Croom. Two toll plazas are located on the mainline and Clonmacken Link. A location plan showing the extent of the project and associated major structures is given in Figure 1. This paper relates only to the approach road earthworks to the north and south of an immersed tube tunnel beneath the River Shannon. It excludes the tunnel and its associated float out and casting basin structures.

Much of the 10 km long roadway is on embankment to maintain the road above potential flood levels and for crossings of existing creeks, roads and railways. A number of flood bunds are incorporated into the highway scheme to provide continuity of the existing river flood protection and to protect the roadway and tunnel. For earthworks design purposes the mainline reference chainage commences at zero at Rossbrien Interchange.

SITE CHARACTERISATION & ALLUVIUM PROPERTIES

As depicted on the cover photograph, the route passes through flat, low lying alluvial flood plains of the River Shannon and its tributaries named Ballinacurra, Meelick and Cratloe Creeks, which are all tidally influenced. Ground levels generally vary from +1 to +4m OD but locally rise to +13m OD at a small hill at Ballykeefe plus the two tie in locations at the project limits. The route crosses a man made water body named Bunlucky Lake which was historically created by excavating clay soils required for cement production and later used for disposal of waste slurry materials from the nearby Irish Cement Plant.

The flood plain of the River Shannon is underlain by extensive deposits of very soft to soft alluvium comprising mainly organic silt / clay to depths typically up to 13m. South of the Shannon the alluvium is present over only 60% of the mainline and varies in thickness from 1 to 6.5m. In the vicinity of St. Neesan’s Road (Bridge B04) the alluvium is overlain or has been locally replaced by Made Ground. North of the Shannon the alluvium is present over 90% of the alignment and typically varies in thickness from 3 to 13m. Greatest thickness of alluvium occurs north of the mainline Toll Plaza towards Bridge B09 Meelick Creek, in the vicinity of Clonmacken Link Toll Plaza and exceptionally is 16m deep at the southern end of the Casting Basin where the road is in cutting.

Alluvium soils contain isolated layers, or pockets, of highly organic soils and peat. These layers of increased organic content are up to 2.5m thick but more typically 1m or less. The alluvium sediments are underlain by deposits of predominantly fine grained glacial tills with occasional coarse grained layers and/or limestone.

A supplemental site investigation was designed and implemented in 2006 to augment the existing factual and interpretive geotechnical reports. The primary goals were:

- provide additional coverage to confirm stratigraphic variations;
- provide more data on undrained strengths derived from CPTu cone penetrometer testing;
- obtain high quality piston tube samples to assess design parameters for consolidation, creep and undrained strength ratio;
- confirm design parameters for cohesive fill soils.

A brief summary of the engineering properties of the soft alluvium follows but a much more extensive description is given in Buggy & Peters (2007).

Index Properties.

The uppermost 1m, approximately, of alluvial material is a firm to stiff desiccated “crust” overlying very soft to soft, grey, silt with organic material or uncompact grey silt with abundant organic material. The stratum occasionally contains bands of more sandy material or shell fragments but is generally free of distinct laminations and partings. Classification test data are presented in Figures 2 and 3. Organic content measured by loss on ignition typically varies between 2 and 10 % but exceptionally is up to 34% in peaty layers.

Undrained Strength.

Undrained strength of the alluvium deposits were determined from the following methods:

- Insitu Cone Penetration Testing ;
- Insitu borehole vane testing;
- Undrained triaxial (UU) tests in the laboratory;

Figure 4 shows the comparison of the undrained shear strengths derived from CPT tests using an N63 value of 17 as defined by Bihs et al (2010); insitu vane tests (with Bjerrum correction for Plasticity Index); and laboratory undrained triaxial tests for two nearby locations.
The ratio of undrained shear strength to effective vertical overburden stress $c_u / p_{0\varepsilon}$ has a controlling influence upon the short term stability of multi-stage embankments constructed upon soft alluvium foundation soils. At Limerick the ratio was interpreted from different laboratory test methods including the following:

- CAUC triaxial compression tests;
- CAUC triaxial extension tests;
- Direct Simple Shear (DSS) tests.

Figure 5 summarises the test data with the ration being greatest for triaxial compression tests and least for triaxial extension tests. A mean of all three tests is considered to be close to the operational conditions prevailing along the potential failure plane beneath a typical embankment and this would produce a mean ratio of 0.29.

**Drained Strength**

KoCU triaxial tests were performed with pore pressure measurement so that effective stress conditions could be ascertained at any time during the shearing stage of the test. The angle of drained shearing resistance as measured in various tests from different phases of site investigation showed considerable scatter with little evidence of any trend when plotted against Plasticity Index. A conservative design value of 28° was adopted in alluvium.

**Consolidation Characteristics & Stress History**

Figure 6 presents the virgin compression ratio, $C_v / (1 + e_v)$, derived from the final settlements after 24 hour consolidation tests, plotted against moisture content. The data collated from the site compares well to the empirical relationships derived from testing of soft soils. (Ref: Simons, 1974 and Eide & Holmberg, 1972).

The coefficient of radial consolidation ($C_{vr}$) may be evaluated by several means including: obtaining $C_{vr}$ from lab consolidation tests; deriving $C_{vr}$ from field tests such as CPT dissipation tests or field permeability tests; and finally back calculation from instrumented field case histories in similar soils. Figure 7 presents the results of standard oedometer testing for the rate of consolidation, $C_v$, plotted against the mean effective stress estimated from the in situ vertical stress plus 100 kPa equivalent to a 5m embankment height load. A range of $C_v$ from 0.5 to 2.5 m²/yr was derived from these tests and a design value of 1.0 m²/yr was selected based on laboratory testing as well as back analysis of $C_{vr}$ from nearby projects. OCR values were determined from the laboratory oedometer tests using the classical Casagrande construction. They varied from 1.5 to 3 in the upper crust while below 1.5 m depth the derived OCR typically varied between 1.0 and 1.5.

**Secondary Creep Ratio**

Figure 8 presents the secondary compression ratio $C_{sp}$ (defined as change in strain per log cycle of time) derived from oedometer testing carried out during the ground investigation. The data compares well with the empirical relationship postulated by Simons (1999) below.

$$C_{sp}(NC) = 0.00018 \times mc(\%)$$

**CASE HISTORY DATA**

A number of reported case histories from projects involving embankment construction in similar soils in Ireland was reviewed and the relevant design parameters and construction details are summarised in Table 1. The projects of greatest significance to the Limerick Tunnel PPP due to their proximity and hence similar construction in the same alluvium deposit were:

- N18 Bunratty Bypass, Co. Clare;
- North Approach Mallow St. Bridge, Limerick;
- Bunlicky WWTP, Dock Rd. & Corcanree Pumping Station, Limerick Main Drainage.

Further details of the schemes are given Buggy & Peters (2007) and project locations are shown on Figure 9.

Of particular note are the following observations:

- There has been good local experience with the use of surcharged embankments combined with vertical drains and geosynthetic basal reinforcement as an effective soil improvement technique in Limerick area since the 1980’s.
- Historical road contracts employed relatively slow filling rates of 0.1 to 0.25m / week plus surcharge durations of up to 2 years. Higher filling rates and surcharge periods of 1 year were successfully achieved at Bunlicky WWTP.
- Coefficient of Consolidation $C_v$ deduced from back analysis of nearby sites typically ranged from 0.5 to 2.5 m²/yr which agrees well with laboratory test data obtained for this project.
- The ratio of undrained shear strength to vertical effective stress $c_u / p_{\varepsilon}$ typically varies between 0.25 and 0.4 in similar organic alluvium soils as derived from differing testing methods.
- Large primary consolidation and long term secondary creep settlements of up to 1.4m and 0.3m respectively have been observed in similar project conditions.
GEOTECHNICAL DESIGN

GENERAL OUTLINE OF SOLUTIONS

Principle methods adopted for earthworks along the project include one of more of the following ground improvement or slope stabilisation solutions:

- Full or partial excavation and replacement;
- Prefabricated Vertical Drainage (PVD);
- Geosynthetic Basal Reinforcement;
- Multi-Stage Construction Techniques
- Surcharging
- Rock Fill Causeway Construction.
- Rock Cutting and Rock Bolt Stabilisation.

The primary technical factors determining which method to adopt were: the depth of soft alluvium; height of road embankment; time available for construction; proximity to structures / utilities; and settlement tolerance criteria. Economic considerations dictated that alluvium below 4 m depth was typically not excavated due to stability concerns and the increased costs of temporary works, importation of backfill and disposal of unsuitable soils. Some exceptions to this limit depth occurred at bridge structure transition approaches, the N18 Interchange and within the Casting Basin where the required excavation depth was over 10m to permit tunnel segment construction.

Figure 10 shows the distribution of the various selected earthworks solutions along the route and it is noteworthy that there is such large variation in adopted designs for a relatively small project length. As the depth of soft soils generally exceeded 3 m and since sufficient time existed in the construction programme, vertical drainage and surcharge measures were selected as the most economic method to accelerate consolidation settlement and decrease secondary creep to within acceptable limits. This solution was adopted for about 6 km of the route.

PRIMARY CONSOLIDATION AND PVD DESIGN.

The estimation of primary consolidation settlement magnitude and rate was performed assuming classical theory and using compressibility and stress history parameters $C_{c}$, $C_{v}$, $C_{s}$ and OCR derived from laboratory test data and supported by relevant case histories with back analysed field parameters. In this simplistic approach the primary and secondary settlements are decoupled and assumed to progress sequentially. In reality this an approximation of true soil behavior as pointed out by many researchers but the validity of this approach to several case histories in Irish soft soils has been adequately demonstrated by Farrell (2000) including for a site in Limerick.

The correlation of Compression Ratio $C_c/(1+C_s)$ to natural moisture content shown in Figure 6 permitted settlement estimates to progress by first producing a graph of natural moisture content variation with depth for relatively short sections of the route, typically of around 200m length. In sections where the alluvium was to be improved by means of surcharge and vertical drains, about 30 of these graphs were developed and a cautious design profile for moisture content versus depth was selected slightly above the mean. From this design profile and assuming site wide parameters for OCR in the surface crust and deeper alluvium deposits of 10 & 1.2 respectively plus a mean ratio for $C_c/C_s$ of 0.06 based on lab testing, an estimate for primary consolidation settlement in the alluvium could be made. Settlements in the glacial soils were estimated by adoption of compressibility derived from SPT N values but were very small (under 5%) by comparison to the compression in the alluvium.

This simple approach offers many advantages including:

- Ability to reflect local variations in compressibility through moisture content tests which are much more frequently available along the project route;
- Use of an average compressibility trend line avoids skewing the settlement estimate due to anomalous oedometer data obtained at a particular borehole location.

Time rates for primary consolidation due to radial drainage were estimated by the equation developed by Barron (1948). As discussed in previous section of this paper a range of $C_s$ values from 0.5 to 2.5 m²/year were obtained from both laboratory testing and case history back analysis in similar deposits in the Shannon Estuary. A cautious design value of $C_s = 1$ m²/year was adopted for radial drainage and the contribution of vertical drainage was ignored.

Vertical drains consisting of Mebradrain MD7007 were typically installed at 1.3 m c/c triangular spacings to give a theoretical radial drainage time of 19 months to achieve 95% radial drainage. In limited high fill areas at the southern approach to Bridge B11, drains were installed at 1.0 m c/c spacings. A trial embankment was designed with drains at increased spacings of 1.5 and 1.8m c/c in non critical areas of embankment supporting car parking at the Clonmacken Link Toll Admin Building. The purpose was to investigate if the use of wider drain spacings could be
adopted while still meeting the construction programme but this proved not to be possible.

Drains were installed through a blanket of National Roads Authority (NRA) Specification Class 6C fill with a CI 609 geotextile separator below and above. In many areas the road embankment also served as a flood defence berm and so the PVD blanket did not extend over the full toe width of the embankment but was terminated at a point where a 1:1 line from the crest met the ground or vertically under the outer flood berm crest. At both Toll Plazas an outer flood defence berm protected the roadway which was constructed to a lower level and PVD extended to the outer flood berm crest. Figure 11 shows the typical details adopted. The total number and length of vertical drains installed was 200,000 and 1,430,000 m respectively giving an average drain length of 7.2m.

SECONDARY CREEP AND SURCHARGE DESIGN

Surcharge has multiple benefits concerning the deformation performance of embankments constructed on soft foundations soils. Firstly surcharge increases the total stresses applied to the foundation soil and thereby increases the amount of consolidation drainage and settlement at a given time. Following surcharge removal, after sufficient time has elapsed, the amount of primary consolidation settlement remaining under the permanent load is either greatly reduced or eliminated. The second benefit derives from a reduction in the rate of secondary creep which occurs following surcharge removal but this only happens if the surcharge is maintained long enough to develop an effective overstress in the foundation soils significantly above that due to the permanent embankment load. Additionally there is a lag, or delay, in the onset of secondary creep following surcharge removal.

These benefits have been studied and quantified by reference to several well documented case histories by Ladd, (1989), Ng, (1998), Nash & Ryde (2001) and Mesri & Castro, (1987). A summary of Ladd’s empirical approach is indicated in Figures 12 and 13. Ladd compared reductions in the rate of secondary compression $C_{v0}$ (following surcharging) to the normally consolidated rate $C_{v0}$ (NC) (without surcharge). The degree of reduction, or improvement ratio $C_{v0} / C_{v0}$ (NC), depends on the degree of ‘over-consolidation’ achieved by use of a surcharge. The improvement ratio $C_{v0} / C_{v0}$ (NC) was related to a parameter called Adjusted Amount of Surcharge (AAOS).

$$AAOS = (p^* - p'_f) / p'_f \text{ (expressed as a percentage)}$$

where $p^*_f$ = maximum effective stress during surcharge fill placement 
$p'_f$ = final effective stress following surcharge removal

Note that the term P’s is given in terms of effective stresses achieved during surcharging and not the initial total stress applied. Thus losses in filling heights due to settlement must be allowed for and additional fill is required to maintain the degree of overstress as designed.

Ladd’s approach and case history data was not well documented by experience in UK or Irish soils, so a series of long term oedometer tests were included in the supplemental SI in 2006 to attempt to validate it. Only a few such tests could be performed and the results indicated very high improvement ratios. Research was subsequently undertaken at UCD to verify if the behaviour of alluvium at Limerick would be similar and the results as published by Conroy et al (2010) are summarised in Figure 14. Although these data were not available to inform the original design, it is at least gratifying that they match well to Ladd’s mean line based on a world wide data base. More recently a case history from surcharged embankments in Hamburg Germany has revealed a very similar improvement in creep behaviour based on 6 years of field data (Chaumet et al., 2011).

To achieve a target reduction in creep settlement or improvement ratio, it follows that the surcharge load must be set at a sufficiently high proportion of the permanent embankment load. Thus for the same degree of improvement the surcharge height must increase as the embankment height increases. For a fixed surcharge and final embankment height the AAOS and hence degree of improvement reduces with depth in the compressible layer as the final vertical effective stress increases with depth. Thus the degree of improvement is greater for the shallow soil layers compared to those deeper. A target AAOS of between 30 to 40% was selected as being ideal and was typically met by surcharge heights between 2.0 and 2.5m. AAOS in excess of 40% was deemed to have a limiting improvement ratio of 0.1, although both Ladd and UCD research data suggests than even greater improvement may be possible up to 50 or 60 % AAOS. On occasion the combination of high embankments over deep alluvium deposits reduced the AAOS in the lower layers to 25%. In order to meet more stringent settlement criteria at bridge transitions, the surcharge height was locally increased to a maximum of 4m.
Without surcharge, secondary creep settlement of embankments over 35 years following construction would typically vary from 100 to 300mm and this would not comply with the project performance specifications. Following the adoption of surcharge the predicted creep settlements over the same time duration were typically reduced to between 20 and 50mm.

**BRIDGE APPROACH TRANSITION DETAILS**

Three bridge structures carrying the mainline road over tributary creeks were located within areas of deep alluvium where full excavation of alluvium was not economically feasible. This lead to concerns about differential settlements between the structures supported on driven steel H section piles founded in the Limestone bedrock and the approach embankment on alluvium. Such solutions were adopted at culverts or underpass structures which were adopted in combination with each other at a typical height up to 3.25m. A typ example of how the ground response of pile supported embankments is provided in an embankment over 35 years.

A range of solutions were adopted at the approaches to these structures as follows:

- Short sections of pile supported embankments were constructed typically 15 – 20 m distant from the bridge abutment. In some cases these sections also encompassed culvert structures or farm access underpasses;
- Full or partial local excavation and replacement of soft alluvium up to 5m depth and extending to 15 m from bridge abutments within sheeted, braced cofferdams. A sloping transition of partially excavated soft soil at 2:1 (H:V) then extended away from the bridge;
- Locally increased surcharge heights up to 3.25 - 4m above a partial excavation sloped transition, if present, and extending from 40 up to 100m from the bridge;
- Geosynthetic reinforcement consisting of Basaltex 200/50 laid longitudinal parallel to the road at the embankment base as a mitigation to reduce local differential settlements. The woven geotextile was encapsulated by graded granular fill layers above the drainage layer, anchored at the structure either above a piled slab or by a return lap and extending at least 10 m beyond the sloped partial excavation transition;
- A typical example of how the various solutions were adopted in combination with each other at a typical structure is provided in Figure 15. No special details were adopted at culverts or underpass structures which were supported by spread footings bearing upon surcharge improved ground. Such structures were constructed after completion of the ground improvement programme.

**MULTI-STAGE EMBANKMENT STABILITY**

It was readily apparent both from local earthworks experience and from preliminary calculations that a single stage embankment quickly constructed above the soft alluvium with side slopes of 2:1 (H:V) would fail before reaching heights of around 3m. This fact determined that a staged construction method with extensive instrumentation to monitor the ground response and carefully controlled rates of loading (filling) would be required in order to achieve success. The supplemental site investigation was specifically designed to provide information on the design parameters required to support this design approach.

O’Riordan and Seaman (1994) give a brief overview of the design methodologies that can be adopted for multi-stage embankments. The designers adopted the undrained strength analysis approach as developed by Ladd (1991). This employs a normalized undrained strength ratio $c_u / p_0$ to predict the operational shear strengths that either currently apply or that would apply at some future time after initial loading based on the estimated (or measured) partial consolidation and pore pressure conditions of the layer of soil in question. Stability at any stage of construction was evaluated by limit equilibrium methods using Bishops Modified Method for circular and Janbu’s Method for block shaped failure planes, the most critical of either being adopted in design. A minimum operating Factor of Safety of 1.25 was adopted for short term loading conditions assuming that the embankment was fully instrumented. In the long term fully drained condition a Factor of Safety of 1.3 was selected for design. The global safety factors adopted in design in 2006 predate the implementation of Eurocode EN1997.

The design process for multi-stage embankments in areas of improvement by surcharge involved several steps as follows:

1. An initial design undrained shear strength $c_u$ profile versus depth was developed for the soft alluvium primarily based on CPT data correlated to available CAUC triaxial strength data. Typically several $c_u$ profiles were developed in each of the eight Design Units that required ground improvement by surcharge to account for the variability in soil conditions and each profile divided the alluvium and other soils or fill materials into several discrete layers;
2. For each profile the maximum initial height of embankment fill that could be safely constructed
was determined followed by an initial hold period (typically 6–8 weeks duration);

3. The degree of consolidation attained at the end of the stage 1 hold was estimated as previously explained for the vertical drain spacing adopted. No consolidation or strength increase was conservatively assumed for alluvium located outside of the PVD zone;

4. The new vertical effective stresses were calculated for these conditions and the operational undrained shear strengths estimated for individual soil layers. No increase in undrained shear strength was adopted if the new vertical effective stress did not exceed the estimated preconsolidation pressure $p_{oc}$;

5. Steps 3 & 4 were repeated for additional fill heights in further stages as required to achieve the desired maximum embankment height including temporary surcharge;

If the total filling duration to achieve maximum height was deemed excessive, typically in excess of 6 to 9 months depending on the Contractor’s programme, then the use of basal geosynthetic reinforcement was considered to increase the temporary stability and thereby reduce the total time required for initial filling to full height. Basal reinforcement was required for approximately 1.7 km north of the Shannon or 28% of the road embankment requiring surcharge typically where the total temporary embankment height (including surcharge fill) exceeded 6 m at the approaches to bridge and underpass structures. Reinforcement was typically Basetex grades 400/50 to 600/50 kN/m tensile strength but design strengths assessed for stability analysis were appropriately reduced in accordance with BS8006:1995) to reflect short or long term factors such as construction damage, manufacture variation, environmental degradation and creep effects.

The significant role played by the embankment fill materials in the critical short term stability of embankments should be recognized. The lower sections of embankments below +5.2 m OD and in the majority of cases all of the embankment fill height including temporary surcharge were constructed from an NRA Class 2C stoney cohesive fill derived from local sources of glacial till. This material was specified to have a minimum undrained strength of 75 kPa as determined by UU triaxial tests on 100 mm diameter samples compacted under 2.5 kg Standard Proctor rammer to BS1377: Part 4 while meeting the minimum acceptable MCV which varied from 7 to 9 dependent upon the source. In locations where the road embankment also served as a flood protection berm a maximum permeability of 1 E-08 m/sec was specified. A number of preliminary tests were performed on fill materials from various sources to confirm that these lower and upper bound targets could be reliably met.

This cohesive fill material type exhibits an undrained strength which is much superior to a granular rock fill’s frictional shear strength for embankment fill heights up to 8 m. The possibility of shrinkage cracking and subsequent water filling of the cracks was considered but not adopted in undrained stability analysis based on the following considerations:

- Climatic conditions in SW Ireland would rarely result in extended drought periods likely to cause extensive deep cracking of this fill type;
- Glacial Till fill materials typically exhibit relatively low PI of < 20 and so their volumetric change due to moisture loss is not very large;
- If drought conditions were to prevail, the Contractor could easily adopt mitigations such as wetting and recompack of the fill surface.

There were occasions when the Contractor sought to substitute granular rock fills for cohesive materials in the upper sections of embankments, mostly in an effort to better balance the fill material reuse when temporary surcharge was removed. Typically this design change required either longer hold durations and total filling periods or the addition of geosynthetic reinforcement in the embankment. Unusually the rock fill embankment constructed in Bunlucky Lake consisted of a lower section of well graded granular Class 6A granular material loose dumped to 1 m above the water line below an upper section constructed from Class 2C cohesive fill. The construction of this causeway is described in more detail later in the paper but it is important to note that it was designed as a two stage embankment with no soft ground foundation present, unlike the multi-stage sections on surcharge and PVD improved alluvium.

Multi-stage embankment construction designs were summarized in tabular format for each design profile and the earthwork drawings also reflected the reinforcement and stage hold durations. Earthworks construction was controlled in the field by careful review of instrumentation data by the designer’s site staff and the filling schedules and hold periods were altered to reflect the true soil behavior as monitored by field instrumentation.

Jardine (2002 & 2006) gives an excellent summary of the behaviour of multi-stage embankments constructed on soft
foundation soils. Based on a number of fully instrumented and well documented case histories he notes the following key principles of their behaviour which can be of use in monitoring performance and assessing stability:

1. Large ground movements due to volume changes can occur as instability is approached;
2. Instability is primarily related to lateral spreading of the foundation and this can be monitored by assessing deformation ratios of lateral movement at the toe $\Delta Y$ to maximum settlement at the crest $\Delta S$ (see Figure 16). The limit criteria for such ratios will be different for single compared to multi-stage embankments and indeed will vary with each site due to soil material properties, embankment geometry, soil profile and loading rate.
3. Similarly as instability is approached the ratio of pore pressure change in the foundation soils to increased total loading approaches and exceeds unity;
4. An observational approach is only valid if adequate instrumentation and a degree of redundancy due to loss is provided. The time necessary to acquire, process, evaluate and provide a control response must also be sufficiently short to avert a failure.

Limited case history experience from Queenborough and Sandwich, two multi-stage embankment sites with low permeability clays in the UK cited by Jardine, suggested that safe deformation ratios $(\Delta Y/\Delta S)$ were under 0.22, but note these cases both related to foundation soils with $c_u / p_o$ ratios $< 0.23$ while Limerick alluvium exhibits much higher undrained strength ratio of 0.29. Jardine concludes that for multi-stage embankments stability will be acceptable if the global trend for deformation ratio $(\Delta Y/\Delta S)$ is less than 0.3 but notes that this might vary for other soil types including peats. Farrell & Davitt (1996) collected some limited data for the nearby N18 Bunratty Bypass Overbridge during construction which seemed to confirm that a limit value of 0.3 would be acceptable for unreinforced embankments and that lower ratios of 0.2 could be expected for basal reinforced embankments.

Because the designers would place great reliance upon the monitoring data to inform them of impending unsafe conditions, the selection of appropriate threshold limits for the deformation ratio $(\Delta Y/\Delta S)$ was critical. Some further FEM analysis was performed at two cross sections Ch 2+850 & 8+300 m deemed to be representative of typical conditions south and north respectively of the Shannon. Constitutive models used for soft alluvium included standard isotropic soft clay model in the PLAXIS suite plus anisotropic models S-CLAY1 and ACM which was performed by University of Strathclyde. Further details of the anisotropic model parameters and results are given by Kamrat-Pietraszewska, et al (2008). The FEM results suggested that the maximum deformation ratio to be expected for the proposed stage loading schedules at adequate Factors of Safety might range up to 0.6.

The following threshold limits for monitoring data were adopted as indicative of developing failure based on an average filling rate of 0.5 m/week with an absolute prohibition on any single incremental fill rate exceeding 1m/week.

- Incremental pore pressure ratios $\Delta u/\Delta s_o > 1.0$;
- Global Deformation Ratios $(\Delta Y/\Delta S) > 0.5$;
- Deformation Ratios > 0.3 represented warning conditions where fill rates and performance data had to be more closely monitored;
- Incremental change in settlement or toe movement $> 0.1m$ between consecutive readings

INSTRUMENTATION MONITORING

An instrumentation monitoring programme was developed to observe full scale field behaviour during the construction process, provide advance notification of the development of unstable slopes and to ensure that design assumptions were being met. The general objectives were:

- Monitoring of total and differential settlement performance of embankments during construction;
- Monitoring of deformation ratios of single and multi-stage embankments during construction;
- Monitoring of pore pressures within the soft alluvial foundation soils to confirm that excess pore pressures are fully dissipated prior to surcharge load removal;
- Monitoring of pore pressures within critical layers of the soft alluvial foundation soils near critical failure planes under embankment slopes;
- Monitoring of extension strain in geosynthetic basal reinforcement layers

A total of 13 fully instrumented cross sections were selected at representative locations plus near bridge structures where temporary fill heights would be greatest. An average coverage of 1 instrumentation cross section per 500m of surcharge / PVD improved embankment was
thus achieved. A standard instrumentation cross section is shown on Figure 17 and included a pair of settlement plates 3m inset from the embankment crest, survey monuments 1m offset from each toe, VW piezometers arranged at the centre point of the triangular PVD layout under the embankment centreline typically at 3m depth increments plus a single piezometer under both mid slopes at 2 metres depth. At six locations a series of basal reinforcement strain gauges were installed both at mid slope and under the crest on both sides of the embankment. Two magnet extensometers were installed in deep areas of alluvium below the embankment centrelines.

Settlement plates and toe survey monuments were generally arranged in pairs at 50 m c/c spacing along the mainline. Additional plates were installed where the embankment widened at both Toll Plazas to maintain the same coverage. All of these instruments were initially surveyed using total station systems referencing to baseline survey stations located at 500m along the route which were founded on glacial till / bedrock at depth. GPS systems were typically used for surveying during active filling. An initial comparative trial between both survey methods showed the GPS accuracy to be generally within +/- 10mm which was considered adequate. During final surveying of settlement plates and monuments prior to release of surcharge, the surveying method reverted to traditional levelling with greater accuracy of +/- 2mm.

Monitoring frequency was stipulated in the project specifications dependent upon whether the area was being actively filled or not and the current maximum settlement rate. Active areas of filling with settlement rates > 20 mm / week required twice weekly monitoring but daily monitoring was triggered when monitoring threshold limit values described previously were exceeded.

PARTIAL EXCAVATION – N18 INTERCHANGE

A pocket of soft alluvium soil was discovered during the Supplemental Investigations extending up to 9m depth between Ch 9+490 and 9+630m. The location coincided with the northern tie in to the original N18 dual carriageway which was constructed on 2 – 3 m of fill above the alluvium. This interchange was one of the final components of the project to be completed but construction had to be delayed and completed relatively quickly in order to achieve the following goals: i) balance of earthworks materials involving reuse of surcharge fill; ii) permit a redesign of a large viaduct structure B11 and revisions to the road alignment to incorporate an access road underpass; and iii) reduce the traffic management durations along a heavily trafficked, high speed section of the existing N18 to the minimum.

A number of alternatives were considered including the use of lightweight aggregate fill; deep soil mixing; construction of a pile supported embankment; and accelerated surcharge with or without use of vertical drains but these were rejected due to economic cost or programme considerations. The solution adopted was to partially excavate the fill and soft alluvium to a depth of 5m to level -1.5m OD, place a granular drainage blanket, backfill with acceptable general fill and construct embankments up to heights varying from 1 to 4m above the original ground level. This solution left a significant thickness of alluvium in place and was predicted to result in up to 260 mm of long term settlement post construction. Such settlement magnitudes are expected to exceed the construction contract limit values for differential settlement and require the operator to make additional pavement interventions during the concession period.

ROCK CUTTING & STABILISATION AT B06

A short 340m long section of the road at Ballykeefe Boreen (see Figure 1) passes through an 8m maximum depth cutting in glacial till soils 1 – 3m thick overlying limestone bedrock. The section also contains a single span overbridge structure B06 carrying a minor local road which was designed to be supported on shallow spread footings bearing upon rock. Although no karst features were evident from SI boreholes or geologic databases in the vicinity of the cutting, evidence of karst solutioning including sub vertical open cracks up to 40mm wide and sub horizontal (<10 degrees) bedding planes with up to 8mm of clay infill materials were observed during the preliminary excavations of the cutting. Unconfined Compressive Strength testing of rock core showed the intact rock strength to be moderately strong to strong (50 to 200 MPa) while Solid Core Recovery and Rock Quality Designation rapidly increased with depth especially at 1m below the rock head level.

Design parameters of $\phi = 35^\circ$ to $40^\circ$ and $c' = 50$ to 100 kPa were assumed for the general rock mass while distinct clay filled discontinuities were assumed to exhibit lower strengths associated with sandy clay derived from glacial overburden soils. A discontinuity survey was prepared by a Consultant Geologist and bridge foundation levels were selected to avoid the clay seams where possible. A rock bolting system was designed to provide additional support to the section of bedrock beneath the bridge foundation while generally the rock profile was excavated at 2V:1H.
EARTHWORKS CONSTRUCTION

EARTHWORKS QUANTITIES & SEQUENCING

A total volume in excess of 2.8 million m³ of fill was required for the construction of the road embankments on the Limerick Tunnel PPP. This figure included 515,000 m³ of material required to temporarily surcharge the embankments as part of the ground improvement works. The consolidation process resulted in a loss of approximately 310,000 m³ through settlement in areas treated by PVD and surcharge. A summary of the general and processed fill requirements and sources used on the project is presented in Table 2.

Approximately 95% of the earthworks materials required for the scheme was imported from commercial quarries or licensed borrow pits sourced by the Contractor. The remaining 5% was generated in the two cut sections at Ballykeefe and Rossbrien, located on the south of the River Shannon. The locations of major quarries utilised by the project are shown on Figure 18.

The nature of the scheme, combined with the cost, the location and the availability of suitable materials for the construction of the road embankments necessitated significant early interaction between the contractor and the designer. This interaction was critical in ensuring that the contractor’s expectations were aligned with the designer’s objectives.

Excavation of unsuitable material and its replacement with engineered fill was only carried out in areas where it was commercially viable. In many areas however the depth of the soft material was such that the installation of vertical drains combined with surcharge loading was the preferred design solution. In these areas the contractor and the designer set about achieving the maximum use of available material by selecting the optimum combination of vertical drain spacing, surcharge height and surcharge duration all the while ensuring that the expected design performance was achieved within the time constraints of the construction programme.

The successful implementation of these design principles ensured that the Contractor did in fact achieve the optimum reuse of all the available material. Transition arrangements also ensured that the surcharged embankments were sufficiently distanced from the locations of the new structures to allow bridge construction to progress in parallel with the consolidation of the embankments. This was necessary to ensure that the project could be completed in accordance with the target programme.

South of the Shannon

The southern section of the scheme was divided by two tidal tributaries of the River Shannon. The cost of temporarily bridging these rivers was considered prohibitive and it was therefore decided to use Class 2C fill material from the Irish Cement source located west of the Dock Road Interchange for the construction of the road embankments west of Bridge 5 (see Figures 1 & 18).

The use of vertical drainage combined with surcharge as a ground improvement technique was minimised where possible for the construction of the road embankments east of Bridge 5. This section of the scheme, which included the Rossbrien Interchange was constructed using materials imported from sources located in the eastern suburbs of Limerick City. Due to utilising these urban locations, the Contractor set itself an objective to minimise the number of truck movements per day on the public roads in these areas. The excavation of shallow unsuitable soft materials, where possible, eliminated the programme requirement for consolidation thus allowing the construction (fill import) period in these areas to be extended. This in turn facilitated the construction traffic volumes remaining within the limits agreed with the Gardai and the Local Authority. The general principle was to restrict the number of deliveries to each particular section to a maximum number of 15 per hour. Possible delivery routes were assessed and the most suitable options carefully selected with particular regard to health and safety rather than distance or travel time. In addition deliveries during peak traffic were also avoided by scheduling work breaks accordingly.

The location and quality of material available at the Irish Cement Quarry was such that much greater volumes could be delivered daily by using articulated dump trucks. Fortunately Irish Cement had all the relevant and necessary licenses already in place to allow works to commence on the construction of the embankments at the Dock Road interchange in the Autumn of 2006. This was a particularly high embankment which was constructed in two stages with the second stage commencing in the Spring of 2007.

The Winter of 2006 was spent extracting in excess of 350,000 m³ of rock fill at the Irish Cement Works which was required for the construction of the first phase of the causeway across Bunlucky Lake. This section of the scheme was designed to be founded on firm material and
consequently it was necessary to remove all of the soft deposits overlying the proposed formation.

The first phase of the rock causeway across Bunlicky Lake was constructed to a level of +1.00 m O.D (1.0 m above the standing water level in the lake), between February 2007 and April 2007. The remaining build up of the road embankment across Bunlicky Lake was constructed in phase 2 using a Class 2 material which became available once consolidation of the surcharged road embankments between the lake and Bridge 5 was complete.

The road embankments east of Bridge 5, constructed on ground treated with vertical drains, were surcharged using Class 6F rock fill. This decision was based on the fact that if a Class 2 material was used for surcharge there would not be a suitable area for its subsequent use and it would then have to be disposed of off site to a licensed facility. The Class 6F rock fill was placed at a level that would ensure that the depth of material in place post consolidation would be in excess of the requirement for the ground improvement layer in the permanent design situation, thus eliminating a now unnecessary excavation operation. The length of embankment to be surcharged was selected with a view to achieving a balance between the volume of surplus Class 6F surcharge and the requirement for this material in the permanent design of the road embankments east of Bridge 5. One further advantage of this construction methodology was that the placing of the Class 6F surcharge instead of a traditional Class 2 surcharge was less dependent on suitable weather conditions. This work could be carried out during the winter period thus providing greater freedom in the construction programme and allowing the maximum time for consolidation of the embankments. As discussed in the design section, granular fill is not as effective as cohesive fill in terms of short term strength, however the embankment heights in this section were modest and this did not prove to be a critical factor.

North of Shannon

There were no cut sections on the north side of the project and as a result all the construction materials had to be imported. The only licensed options available to the Contractor at the time of contract award were Bobby O’Connell’s quarry at Ardnacrusha and a Roadstone sand pit located at Woodcock Hill, north of Crafole, both located in County Clare. The combined output capacity of these two sources was restricted to a total of 2,500 m³ per day due to conditions imposed under planning regulations. The total construction period allowed for in the target programme for the road embankments necessitated the sourcing of alternative material deposits in order to satisfy the total import fill requirement of 1,250,000 m³ for the construction of the road embankments north of the River Shannon.

Because of this material deficit the Contractor entered into an agreement with a privately operated licensed facility south of the Clonmacken link. This allowed the Contractor to remove suitable material from this site on the understanding that it would be replaced at a later date. A sufficient volume of suitable material was provided, combined with the imported material from the quarry operations to allow the construction of the first phase of the Clonmacken link to be completed before the end of 2006. This was a critical programme requirement particularly in relation to the high approach embankments to Underpass U6 and the Clonmacken toll plaza construction.

The Contractor applied for planning permission in January 2007 to extract rock and suitable clay at three locations to satisfy the material requirement for the scheme north of the River Shannon. Following consultation with Clare County Council planning was granted in June and July 2007.

Two of the sources located on the south west side of the tie-in with the existing N18 provided 460,000 m³ of rock and 390,000 m³ of suitable clays. One of the primary advantages of these sources was the provision and utilisation of a dedicated haul route which facilitated the use of articulated dump trucks thus removing the requirement to use any public roadways.

A third source was located adjacent to the N19 junction with the N18 at Shannon in County Clare. This source was approximately 20 km from the site but it came with a significant advantage in that the route to the site was along the N18 which provided for a safe and efficient delivery cycle. A total of 180,000 m³ was imported from this source using articulated road trailers which yielded a delivery rate of 30,000 m³ per week.

The mainline toll plaza was located on the east side of the Meelick Creek, a tidal tributary of the River Shannon. This was a significant fill area which required a total of 650,000 m³ of material. The Contractor erected a temporary steel bridge to span the 40 m wide tidal creek, which allowed the materials be transported directly from local sources without having to use the existing N18. As the lands made available under the contract did not provide sufficient area, the temporary bridge was constructed on line and the
project programme was accordingly adjusted to schedule the construction of the permanent bridge during the consolidation period of the road embankments.

The Contractor realigned the construction of the proposed casting basin for the tunnel elements to an online construction extending from the end of the north ramp to the Coonagh West Interchange. The knock on effect of this revised construction layout meant that, as the casting basin would not become available until all tunnel elements were immersed, the filling of this section of the road to final profile coincided with the release of the surplus surcharge for the adjacent earthworks embankments.

The surplus surcharge from the Clonmacken Link was returned to the adjacent licensed tip in accordance with the agreement between the parties.

As per the southern side, sections at the northern end of the scheme were surcharged using a Class 6F material which ensured the same advantages accrued regarding the programme, efficiency of material use and weather dependence. As the embankments in these areas were significantly higher than those on the south side of the River Shannon, additional basal reinforcement was incorporated in the design to ensure stability during the various construction stages.

*Particular Construction Techniques Utilised*

The ground improvement techniques employed during the construction of the Limerick Southern Ring Road Phase 2 scheme introduced particular challenges in relation to the existing utilities and drainage networks. The new scheme required the diversion of a number of important arterial sewers, communication networks and power supply lines particularly on the southern side of the River Shannon.

The diversion of services was required to be completed in advance of the main embankment construction and was thus subject to risk of damage during the subsequent consolidation. This risk was overcome by installing the ducting for electrical services and communications using directional drilling techniques which allowed continuous welded ducting to be installed on a curved alignment below the soft soils and into the glacial tills.

Arterial sewers installed during the upgrade of the Limerick Main Drainage network were protected by constructing pile supported reinforced concrete slabs at existing ground level to protect the pipelines from excessive loading and subsequent deflection resulting from the newly constructed road embankments.

The northern section of the works crossed many important arterial drains under the control of the Office of Public Works. The contract required that these drains were kept operational throughout the construction phase as they were critical in the flood protection of the low lying surrounding catchment area. Many of these drains were located in areas of deep and soft underlying soils where the vertical settlement due to consolidation was expected to exceed 1.5 meters. This obviously prohibited the use of temporary concrete or plastic pipes and instead a continuously welded large diameter steel pipe was installed at a pre camber to account for the expected vertical settlements.

*BUNLICKY LAKE CAUSEWAY*

The route between Chainage 4+460 and 5+150m passed through a man made lake excavated for cement production at the Irish Cement plant located at Munget on the western side of Limerick City. The lake extended to an area in excess of 100 hectares, with a depth of fresh water of up to 7m. The property lies within the Shannon Basin Special Area of Conservation (SAC) and is considered to be environmentally sensitive.

*Construction & Design Risks*

The site investigation confirmed the presence of between 2m and 3m of very soft silts ($c_u < 20$ kPa) deposited during the cement manufacture process. The original construction concept incorporated in the tender was to displace the soft material using the weight of the embankment material plus temporary surcharge to induce a controlled bearing capacity failure to displace the soft sediments. This concept was reviewed by the construction joint venture and a number of construction, financial and design quality risks were identified as follows:

- Causing a failure to the flood embankment along the southern bank of the river Shannon. The construction concept would result in a significant volume of material being pushed in front of new construction as it progressed north. The displaced material could become trapped against the river embankment imparting forces that would be difficult to calculate. The consequence of a breach of the existing flood embankment would be catastrophic.
- Inducing a major slip failure of the lake bed as a result of overloading the formation to cause displacement of the soft silts. If the temporary fill caused a deep slip on either the eastern or western
side of the embankment, it could compromise the permanent embankment stability with limited options of remedy.

- Health & safety risks to construction workers as a result of relatively sudden failure of overloaded embankments which may not be adequately controlled or predicted.

- Significant increase in the quantity of Class 6 rock fill materials that would be required to construct embankment. This was a significant financial risk as all of the Class 6 material was imported. A 10% increase in material from the net dimensions of the embankment would result in an increased requirement of 80,000 tonnes of rock fill.

- Displacement of significant volumes of soft material outside the limits of the lands made available and the costs of rectifying any trespass.

- Environmental damage to the eco-system in the lake as a result of the nature of the construction concept which loaded the soft material to ultimate failure resulting in large sudden displacements and potential sediment suspension.

- Uncertainty of the proposed construction method, final quality and the resulting risk of unsuitable material becoming trapped under the rock fill embankment compromising the future performance of the highway or failing to meet contractor’s designer and client specified standards.

**Adopted Construction Methodology**

The Construction Joint Venture (CJV) and the earthworks designer developed a construction technique that mitigated the risks posed by the displacement technique. The contractor carried out a series of probes along the eastern and western limits of the new embankment construction. This allowed the designers to accurately estimate the position of the toe of the embankments and the height of Class 6 fill required. A plan and cross section of the adopted design showing the construction sequence is presented in Figures 19 & 20.

The construction technique developed was based on the principles of the controlled excavation of soft soils and placing of rock causeways along the outer extremities of the new embankment. The soft material was removed using an 18 m long reach excavator which placed the excavated soft material against the outer side slope of the rock causeway that was already constructed. The two outer rock causeways were sequentially linked at 70 m intervals thus ensuring that all of the areas of the enclosed cell could be excavated using the long reach plant. The cellular construction also ensured that there was no build up of material ahead of the construction and also allowed the individual cells to be completed independently.

The Contractor recorded a level of excavation in advance of placing of the rock fill which was then plotted against the target depths based on the available site investigation logs. The survey information and verification probing was used by the designers to validate that an adequate depth of soft soil was removed and the underlying foundation was competent (typically exhibiting $c_u > 40$ kPa).

The southern access ramp was located between Chainage 5+000 m and Ch. 5+150 m. The construction methodology used to construct the causeway was further adapted to incorporate a 150 mm crushed rock in the eastern and western outer embankments. This decision facilitated the installation of steel sheet piles around the perimeter of the southern access ramp necessary for the excavation and dewatering of this element of the works.

The main causeway was initially constructed to a level of 1m above water level between January and April 2007, the final analysis of the material used confirmed that the actual volume of material used was 115% of the net volume of the embankment profile. The second construction phase involved the placing of the remaining Class 2C clay to raise the embankment to finished levels. A photograph of the causeway at the end of Phase 1 construction is presented in Figure 21.

**Design Verification**

One of the major risks identified was that the embankment would not meet the settlement performance requirements of the client or the designers. This risk was mitigated by the following measures:

- Monitoring of the “as built” rock fill profile by surveying post construction depths plus settlements over an initial 6 month period after initial filling and again upon completion to full embankment fill height;

- Inclusion of a 6 month minimum hold period in the construction programme, to increase embankment temporary stability as pore pressures dissipated and which also permitted construction of four large concrete culverts;
• The hold period potentially could also be used to accelerate the progression of excessive settlement by use of surcharge;

EMBANKMENT PERFORMANCE

CONSOLIDATION SETTLEMENT & RATE

Typical pore pressure and settlement histories for two instrumented locations at Ch 4+150 and 8+300 m, south and north of the River Shannon respectively, within areas improved by PVD and surcharge are shown on Figures 22 & 23, (a) & (b). Total fill heights of approximately 11.5 & 7.5 m were achieved above soft alluvium extending to depths of 3 & 9 m. The embankment at Ch 4+150 m was unreinforced while at Ch 8+300m it was reinforced with a Basetex 400/50 basal geosynthetic laid with primary strength transverse to the road alignment.

Settlement plates and piezometers were installed following the initial construction of the lower 0.35m thick drainage layer and PVD. As a result the recorded fill height and associated settlements are a slight underestimate as they do not reflect initial filling and settlement caused by the drainage layer. This is presumed to be minimal due to the slightly over consolidated stress history of the alluvium. Piezometers were installed vertically at the centroid of the triangular PVD layout and thus may overestimate the mean pore pressure in the alluvium caused by fill loading.

Some general comments on embankment behaviour follow:

• Drainage response to the first loading stage is relatively rapid and substantially complete within 3 to 4 months. In contrast later loading stages require 12 to 14 months to dissipate the excess pore pressure which reflects the reduction in coefficient of consolidation Cv at higher stress levels above the preconsolidation pressure;

• The original preconstruction baseline pore pressure levels are never fully recovered as the settlement of the instrument changes its long term equilibrium level. This factor must be adequately reflected in any interpretation of the degree of consolidation achieved from the piezometer data;

• Figure 22 (a) at Ch 4+150 m. While the settlement record stops at the removal of surcharge fill in May 2008, the piezometers continued to be recorded and reflect a negative pore pressure response to unloading (as expected) when the surcharge fill is removed followed by a recovery to equilibrium levels.

The measured loading and consolidation drainage durations interpreted from the piezometer data are summarised in Table 3. A single mean coefficient of radial consolidation Cv has been derived based on back calculation assuming the method derived by Barron (1948) and ignoring the effects of vertical drainage. In cases where the alluvium was deep and several piezometers were installed at incremental depths, it was noted that the drainage rate was slower at greater depths and this is again reflects the reduction in Cv with increasing stress level. The range of back calculated radial coefficients of consolidation Cv is from 0.8 to 1.5 m$^2$/year and the values for south of the Shannon being typically slightly above 1.0 and the reverse being true for sections north of the Shannon. There is reasonable agreement with the selected design value of 1.0 m$^2$/year and the nearby case histories.

The distribution of maximum settlement throughout areas of the project improved by PVD and surcharge is shown in Figure 24 (a), (b) & (c) for the mainline embankments south and north of the River Shannon plus Clonmacken Link respectively. Settlement estimates prepared during the detailed design at specific locations by the methods previously described are also shown for comparison. In general there is reasonable agreement between predictions and actual measured settlements typically within 4/- 30 percent. A notable exception being Design Unit 6 located between Dock Road and Bunlicky Lake where the design estimates were an overprediction. It is surmised that this may have been due to the effect of local overconsolidation of the alluvium during historical clay extraction and dewatering of Bunlicky Lake. The somewhat lower moisture content and higher strength profile in DU6 compared to DU4 both attest to this possibility plus the observed higher Cv value at initial filling previously discussed.

DEFORMATION RATIO & STABILITY

Typical deformation ratio histories and inclinometer data for instrumented locations at Ch 4+150 and 8+300 m are shown on Figures 22 & 23, (c) & (d). Some general comments on observed embankment behaviour follow:

• During initial filling to heights of 4 m & 3 m respectively, the deformation ratios rapidly rise to local maximum values of 0.4 & 0.7 at Ch 4+150 & 8+300 respectively. The ratio then
reduces to below 0.3 as settlement continues under constant load before increasing again during the next filling stage but remaining near or below 0.3. This behaviour was commonly observed elsewhere and may be related to the decision to omit vertical drains under the outer portion of the side slope in order to maintain the embankment’s ability to effectively serve as a flood protection bund. During the initial filling stage the outer portion of alluvium beneath the embankment slope has a dominant role in stability but this part of the embankment foundation does not gain appreciable strength with time as there are no vertical drains installed there. Neither does any basal reinforcement extend into this zone. During later filling stages, most of the critical failure plane passes within areas of alluvium with PVD which has benefited from drainage and strength gain over time.

- As shown on Figure 22 (a) the pore pressure response to loading at Ch 4+150 m indicates increases which are around 75% of the total stress increase due to a 6m stage 2 fill height. This suggests that the embankment is adequately stable, consistent with the deformation ratio which remains below the adopted design threshold level of 0.5. In contrast Figure 23 (a) at Ch 8+300 indicates much higher pore pressure responses of near 1.0 times the total stress increase due to the initial 3m fill stage and this has resulted in greater deformation ratios in excess of 0.5 indicative of marginal stability. It is noteworthy that the initial filling rate at Ch 8+300 m was 2.2 m / week, well in excess of the limit design threshold, while at Ch 4+150 m a filling rate of 1.3 m / week was not exceeded during stage 1 filling.

- Figures 22 & 23 (d) indicate the lateral deformation profile with depth determined from inclinometers near the embankment toe. Despite large lateral deformations of around 0.15 and 0.2 m at the ground surface at Ch 4+150 and Ch 8+300 respectively at the end of consolidation, the embankment is more stable at that time. At the critical time period for stability during initial filling lateral movements are far less.

- There appears to be little error for this project in assuming that the maximum lateral deformation occurs at the ground surface and can conveniently be monitored by surface monuments. Data from the Ratty River Bridge case history by Farrell et al (1996) suggests that the maximum lateral deformation may occur a few metres below the surface. Inclinometers are however valuable in confirming the distribution with depth and in identifying local zones of high shear strain possibly related to incipient failure. The time required to read the instrument and analyse the data is a disadvantage however.

The distribution of measured peak deformation ratio throughout areas of the project improved by PVD and surcharge is shown in Figure 25 (a), (b) & (c) for the mainline embankment south and north of the River Shannon plus Clonmacken Link respectively. Where ratios in excess of 0.6 were observed the embankment height corresponding to the peak value is also noted on the graph.

- In general peak deformation ratios south of the Shannon were kept below 0.6. Exceptions occurred at Ch 1+900 & 2+850 m where the embankment was constructed adjacent to and within the original course of Ballinacurra Creek and temporary sheet piling had to be employed to maintain stability.

- Within the mainline north of the Shannon peak deformation ratios were typically below 0.5. Exceptional values in excess of 1.0 occurred near Ch 7+550 m & 7+650 where local failure of an existing back drain behind the Shannon flood protection bund and instability in an area where the surface crust had been historically removed for brick manufacture. In both cases local instability occurred at relatively low embankment heights and was mitigated by culverting the drain and incorporation of geogrid reinforcement over short sections of the outer embankment slope. Other high values occurred near the locations of cross ditches.

- Along Clonmacken Link several values in excess of 1 occurred associated with failures which are described in more detail below.

- Interestingly the occurrence of high deformation ratios is not particularly correlated to maximum embankment height nor to the inclusion of basal geosynthetic reinforcement within the embankment.

EMBANKMENT FAILURES IN CLONMACKEN LINK

During the period from June to September 2007 two significant failures occurred over 50m lengths of embankment (from Ch 140 to 190 and from Ch 780 to 830m ) as filling extended their height to around 3 - 3.5m.
Both failures occurred on the left (northern) side of the embankment and displaced a relatively shallow block of alluvium up to 2.5m deep at the toe outwards and closing a ditch constructed from 5 to 15m distant from the toe. In both cases the failure plane was restricted to the outermost section of embankment and did not pass through the basal reinforcement (present at Ch 140 – 190m only) or the drainage blanket which appeared to function normally based on the nearest piezometer records.

Forensic investigations of both failures revealed the following factors which had contributed to the instability:

- Accidental over steepening of side slopes to around 1:1.3 in lieu of the design slope of 1:2 (V:H);
- Presence of nearby ditches, especially drains cut skew or transverse to the embankment;
- Poor quality fill (Moisture Condition Value 5 to 8 with significant organics) compounded by wet weather conditions (Ch 140 – 190 m only);
- High filling rates around 1.5 to 3 m/week (Ch 140 – 190 m only).

Typical settlement and deformation ratio histories for instrumented locations at Ch 0+150 m are shown on Figures 26, (a) & (b). The large incremental settlements > 0.1m observed in early to mid September 2007 plus deformation ratio approaching 1.0 immediately prior to failure in mid September both validate the selection of design threshold values and were both reliable indicators of future instability. Regrettably the data was not passed quickly enough to engineers who could have acted to avert the failure by preventing further filling after 17 September 2007. The final filling rate of around 3m / week (1.3 metres in 3 days) could never be sustained at this site and served as a salutary warning to all concerned.

Remedial works employed the removal of all materials (both natural alluvium and fill) within the failed soil block; replacement with class 2C fill reinforced with geogrid and the inclusion of a 2m high, 6m wide toe berm over the full failure length to buttress the section.

**BASAL GEOSYNTHETIC REINFORCEMENT**

Basal geosynthetic reinforcement laid transverse to the road alignment was used along 28% of the 6 km route length where PVD and surcharge was employed. The performance was assessed at 6 locations by means of strain gauges attached on both sides of the embankment beneath the crest and mid slope locations. A typical record is presented in Figure 27 and a summary is presented in Table 4.

Typically maximum tensile strains occur under the embankment crest concurrent with maximum settlement and are usually below 3% which is well below the ultimate strain for the material quoted by the manufacturer of 10%. These findings are reasonably consistent with data published by Farrell et al (1996) for the Ratty River Bridge. The initial readings are sometimes negative and require a significant initial fill height of up to 3.5m before the material is stretched suggesting that the materials may not have been adequately tensioned during initial installation. Where a double layer of geosynthetic was employed (due to the presence of a large open drain and exceptionally soft alluvium) both layers appear to have been stressed about equally.

**CONCLUSIONS – LESSONS LEARNT**

Because of the relatively long construction period associated with tunnel construction, surcharge provided an economical solution for ground improvement which had well documented, local case history experience. The available research on benefits of surcharge to creep was critical to its design application and was subsequently validated by both laboratory research at UCD and recent long term field performance in Germany.

Sourcing and licensing local quarries to import nearly 3 million m$^3$ of earthworks material was a major undertaking together with provision of haul roads and temporary bridges to permit proper sequencing of earthworks. Balancing the reuse of surcharge materials with other road construction and refilling of the casting basin was critical to the successful project outcome.

Consolidation settlement and durations were reasonably well predicted by use of relatively simple moisture content profiles and site correlations to compressibility parameters plus back calculated case history data. The decision to eliminate PVD and drainage layer beneath the outer portion of embankment slopes may have had a negative impact to stability at low embankment heights less than 4m. With hindsight this could have been better mitigated by increased local use of a geosynthetic reinforcement such as geogrid, especially in the outer slope areas and in the vicinity of drains near the embankment toe.

Stability of multi-stage embankments are not easily predicted but careful monitoring of filling rates, settlement and lateral deformation ratios at 50 m intervals plus less
frequent measurements of pore pressure response is an effective predictive tool, provided the review and response time is fast enough to control critical loading and prevent instability. Of the many factors influencing stability, filling rate is probably the single most critical to control.

For the particular soil conditions and embankment design geometry applicable to this project, the following critical monitoring threshold values were validated:

- Filling rate < 1 m/week;
- Incremental pore pressure ratios $\Delta u/\Delta \kappa < 1.0$;
- Global deformation ratios $(\Delta Y/\Delta S) < 0.5$ (many embankments were safely constructed however with peak deformation ratios of up to 0.6);
- Incremental change in settlement $\Delta S$ or toe movement $\Delta Y < 0.1m$

It must be clearly stated that these limits do not apply universally to all embankments on soft ground.

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REFERENCES


# Table 1 Comparative Data for Road Embankments on Soft Alluvial / Organic Soils in West Ireland

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<td><strong>Soil Description</strong></td>
<td>Soft organic silt w/ peat layers up to 1m thick. Surficial firm crust up to 1m thick.</td>
<td>Soft alluvial soils</td>
<td>Soft grey organic clay</td>
<td>Soft organic estuarine silt</td>
<td>Organic silty clay Peat Calcareous Marl</td>
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<td>12m</td>
<td>15m</td>
<td>13m (peat layers up to 2.5m)</td>
<td>10m</td>
<td>8m</td>
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<tr>
<td><strong>Natural Moisture (%)</strong></td>
<td>30 – 100 in organic silt 100 – 500 in peat</td>
<td>40 - 70</td>
<td>40 - 80</td>
<td>40 – 120 in organic silt 150 – 300 in peat</td>
<td>100</td>
<td>52 - 64</td>
</tr>
<tr>
<td><strong>Liquid Limit</strong></td>
<td>Similar to natural moisture</td>
<td>-</td>
<td>60</td>
<td>40 – 150 in organic silt 150 – 300 peat</td>
<td>-</td>
<td>47 - 56</td>
</tr>
<tr>
<td><strong>Plasticity Index (%)</strong></td>
<td>-</td>
<td>-</td>
<td>40</td>
<td>30 - 75</td>
<td>-</td>
<td>21 - 23</td>
</tr>
<tr>
<td><strong>Organic Content (%)</strong></td>
<td>42% typical</td>
<td>-</td>
<td>42% typical</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td><strong>Undrained Strength Ratio Cu / Po’</strong></td>
<td>0.3 assumed in design 0.28-0.42 back calculated from field vane test data.</td>
<td>0.5 – 0.6 lab measured 0.25 assumed</td>
<td>0.25 – 0.30</td>
<td>0.30 design 0.20 KoCUE 0.30 DSS 0.36 KoCUC</td>
<td>0.29 – 0.64 0.3 design</td>
<td>0.25 – 0.41 0.3 design</td>
</tr>
<tr>
<td><strong>Coefficient of Secondary Compression C alpha</strong></td>
<td>0.00018 w</td>
<td>0.00018 w</td>
<td>0.016</td>
<td>0.00018 w</td>
<td>0.015 Peat 0.016 organic clay</td>
<td></td>
</tr>
<tr>
<td><strong>Coefficient of Consolidation Cv (m²/yr)</strong></td>
<td>0.35 - 0.5 m²/yr Ch = Cv lab &amp; back calculated from field</td>
<td>1 m²/yr in critical layers Ch = Cv lab</td>
<td>1.5 m²/yr back calculated from field performance</td>
<td>0.5 to 4.0 lab CvH = 0.9 to 1.5 m²/yr back calculated field</td>
<td>CvH = 12 m²/yr</td>
<td>CvV = 0.4 -2.6 m²/yr. CvV=0.5 m²/yr derived from standpipe tests</td>
</tr>
<tr>
<td><strong>Coeff Vol Change Mv (m³/MN)</strong></td>
<td>1.7 – 0.5</td>
<td>1.7 – 0.5</td>
<td>1.7 – 0.5</td>
<td>1.7 – 0.5</td>
<td>1.7 – 0.5</td>
<td>1.7 – 0.5</td>
</tr>
<tr>
<td><strong>Compression Index Cc</strong></td>
<td>0.2 – 0.5 for w&lt;70%</td>
<td>0.35 ave Cc/(1+e0)</td>
<td>0.1 – 0.4 for w = 40 to 120%</td>
<td>0.4 ave Cc/(1+e0)</td>
<td>0.4 ave Cc/(1+e0)</td>
<td></td>
</tr>
<tr>
<td><strong>Max Embankment Ht (m)</strong></td>
<td>9 m excl. surcharge</td>
<td>3m</td>
<td>8.5m</td>
<td>9.5 m max 3 – 5 m typical (excl surcharge)</td>
<td>2m</td>
<td>5.5m</td>
</tr>
<tr>
<td>------------------</td>
<td>-----------------------------------------------------------------------------------</td>
<td>-----------------------------------------------------------------------------------</td>
<td>-------------------------------------------------</td>
<td>------------------------------------------------</td>
<td>---------------------------------</td>
<td>------------------------------------------------</td>
</tr>
<tr>
<td>Filling Duration / Rate (m/wk)</td>
<td>0.18–0.23 base reinforced 0.05 unreinforced</td>
<td>-</td>
<td>-</td>
<td>1 m/week max specified. Actual 0.2 to 0.5 m/week typical in multiple stages.</td>
<td>-</td>
<td>0.1 m/wk</td>
</tr>
<tr>
<td>Surcharge Ht (m)</td>
<td>2m</td>
<td>2m (incl settlement allow)</td>
<td>2 – 4m</td>
<td>0.5m</td>
<td>1m</td>
<td></td>
</tr>
<tr>
<td>Surcharge Duration (month)</td>
<td>Total fill + surcharge duration = 24 months</td>
<td></td>
<td></td>
<td>13 – 18 months actual field duration</td>
<td></td>
<td>3 – 5 months</td>
</tr>
<tr>
<td>Base Reinforcement Layer</td>
<td>Stabilenka 600 in critical sections</td>
<td>Stabilenka 600</td>
<td>N/A</td>
<td>Varies from none to Basetex 200 – 600 kN/m along project</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>Vertical Drain Type &amp; Spacing (m)</td>
<td>0.7 – 0.9m</td>
<td>1.05m up to 40m from bridge &amp; 1.3m elsewhere</td>
<td>Mebradrain 0.8 - 1.1m square grid</td>
<td>Mebradrain MD7007 1.0 - 1.3 m typical triangular grid</td>
<td>Mebradrain 1.4m square grid</td>
<td></td>
</tr>
<tr>
<td>Estimated Settlemnts</td>
<td>1.25m primary, 0.3m secondary in 20 yrs</td>
<td>1m total</td>
<td>Primary 0.4 to 2.2m estimated Primary 0.2 to 2.3 m measured. Secondary &lt; 50 mm in 35 years</td>
<td>3.15m max.</td>
<td>0.85m measured</td>
<td></td>
</tr>
<tr>
<td>Comments</td>
<td>Lateral deformation at embankment toe limited to 0.3 x total settlement for unreinforced &amp; 0.2 for reinforced sections. Geogrid base reinforcement used in low embankment areas. Stability reduced by adjacent river channel. 7mm differential settlement at bridge Ch 8580 from 1991 to 1996. 300m Section with no ground improvement overlaid in 1992 after 250mm settlement. Phi = 32 degrees Cu = 13.5 – 4 kPa Drains required attached to H piles driven adjacent to river bank.</td>
<td>Phi = 28 to 35 degrees KoCU triaxial tests. Cu = 5 to 35 kPa UU txl. Lateral movements of the embankment toe typically 0.1 to 0.6 times vertical settlement (except for local failures).</td>
<td>Phi = 28 to 35 degrees KoCU triaxial tests. Cu = 5 to 35 kPa UU txl. Lateral movements of the embankment toe typically 0.1 to 0.6 times vertical settlement (except for local failures).</td>
<td>Cu = 12 – 26 kPa Data insufficient to back analyse Cvh but performance suggested drainage occurred faster than expected.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 2 Summary of Major Earthworks Types, Quantities and Sources

<table>
<thead>
<tr>
<th>Location</th>
<th>Fill Type</th>
<th>Quantity (m$^3$)</th>
<th>Fill Sources (m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>General Fill Cohesive Glacial Till Class 2C</td>
<td>810,000 50,000 (make up for settlement) 860,000 total</td>
<td>Danninger Site (200,000); Structure B01 Borrow Pit (125,000); Annacoty Business Park (80,000); Recycled Fill from existing Rossbrien Interchange (75,000); Ballykeefe Cutting (40,000); Irish Cement Quarry (340,000)</td>
</tr>
<tr>
<td></td>
<td>General Rock Fill, Class 1C</td>
<td>525,000</td>
<td>Irish Cement Quarry (425,000 – Bunlucky Lake Causeway); Irish Cement Quarry (100,000 – Replacement Fill at Bridge Approaches);</td>
</tr>
<tr>
<td></td>
<td>Processed Rock Fill, Drainage Blanket, Capping / Improvement Layer, Class 6C, 1C(1)</td>
<td>145,000</td>
<td>Irish Cement Quarry (95,000); Ballykeefe Cutting (50,000)</td>
</tr>
<tr>
<td></td>
<td><strong>Sub Total</strong></td>
<td><strong>1,530,000</strong></td>
<td></td>
</tr>
<tr>
<td>North of Shannon Ch 5.9 to 9.7 km + Clonmacken Link (1.3 km)</td>
<td>General Fill Cohesive Glacial Till Class 2C</td>
<td>710,000 260,000 (make up for settlement) 970,000 total</td>
<td>Woodcock hill Quarry (150,000); Brickendon Borrow Pit North of N18 Interchange (320,000); Portdrine Borrow Pit (180,000); Kinehan’s Borrow Pit (N18 Interchange) (70,000); Shannon Business Park (100,000); Clonmacken Tip (150,000)</td>
</tr>
<tr>
<td></td>
<td>General Rock Fill, Class 1C</td>
<td>140,000 (includes Casting Basin Backfill)</td>
<td>Kinehan’s Borrow Pit (N18 Interchange) (100,000); Brickendon Borrow Pit North of N18 Interchange (40,000);</td>
</tr>
<tr>
<td></td>
<td>Processed Rock Fill, Drainage Blanket, Capping / Improvement Layer, Class 6C, 1C(1)</td>
<td>200,000</td>
<td>Kinehan’s Borrow Pit (N18 Interchange) (100,000); Brickendon Borrow Pit North of N18 Interchange (100,000);</td>
</tr>
<tr>
<td></td>
<td><strong>Sub Total</strong></td>
<td><strong>1,310,000</strong></td>
<td></td>
</tr>
<tr>
<td></td>
<td><strong>Project Total</strong></td>
<td><strong>2,840,000</strong></td>
<td></td>
</tr>
</tbody>
</table>

Notes: 1) Quantities exclude earthworks related to Tunnel Approach, Backfill to Tunnel Elements and Dredge Disposal Areas plus pavements sub base and CBM materials.
### Table 3 Summary of Filling Durations and Back Calculated Coefficient of Consolidation (Cv)

<table>
<thead>
<tr>
<th>Chainage (m)</th>
<th>Alluvium Depth (m)</th>
<th>Max. Embankment Height incl. Surcharge (m)</th>
<th>Fill Start Date</th>
<th>Max. Fill Height Date</th>
<th>Surcharge Release Date</th>
<th>Mean Loading Duration (Years)</th>
<th>Estimated mean Coefficient of Consolidation Cv (m²/year)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1+900</td>
<td>4.4</td>
<td>5.8</td>
<td>May 07</td>
<td>Aug 07</td>
<td>Aug 08</td>
<td>1.1</td>
<td>1.5</td>
</tr>
<tr>
<td>2+850</td>
<td>6.9</td>
<td>7.6</td>
<td>Aug 07</td>
<td>Nov 07</td>
<td>Jun 09</td>
<td>1.7</td>
<td>0.9</td>
</tr>
<tr>
<td>3+150</td>
<td>6.4</td>
<td>5.7</td>
<td>Apr 07</td>
<td>Oct 07</td>
<td>Jul 08</td>
<td>1.2</td>
<td>1.2</td>
</tr>
<tr>
<td>4+150</td>
<td>3.0</td>
<td>11.4</td>
<td>Oct 06</td>
<td>May 07</td>
<td>May 08</td>
<td>1.3</td>
<td>1.2</td>
</tr>
<tr>
<td>6+920</td>
<td>9.7</td>
<td>6.9</td>
<td>Aug 07</td>
<td>Feb 08</td>
<td>May 09</td>
<td>1.6</td>
<td>1.0</td>
</tr>
<tr>
<td>7+400</td>
<td>7.4</td>
<td>5.8</td>
<td>Aug 07</td>
<td>May 08 (³)</td>
<td>Mar 09</td>
<td>1.3</td>
<td>0.9 (³)</td>
</tr>
<tr>
<td>8+040</td>
<td>12.0</td>
<td>9.0</td>
<td>Aug 07</td>
<td>Dec 08</td>
<td>Oct 09</td>
<td>1.8</td>
<td>0.9</td>
</tr>
<tr>
<td>8+300</td>
<td>9.0</td>
<td>7.4</td>
<td>Jul 07</td>
<td>Feb 08</td>
<td>Oct 09</td>
<td>2.0</td>
<td>0.8</td>
</tr>
<tr>
<td>8+700</td>
<td>8.3</td>
<td>5.9</td>
<td>Jul 07</td>
<td>Feb 08</td>
<td>Jul 09</td>
<td>1.7</td>
<td>0.9</td>
</tr>
<tr>
<td>CL WD 380</td>
<td>4.0</td>
<td>11.1</td>
<td>Aug 07</td>
<td>Oct 09 (³)</td>
<td>Jan 10</td>
<td>1.3</td>
<td>0.8 (³)</td>
</tr>
<tr>
<td>CL 220</td>
<td>9.2</td>
<td>7.7</td>
<td>Jul 07</td>
<td>Jul 08</td>
<td>Aug 09</td>
<td>1.8</td>
<td>0.9</td>
</tr>
<tr>
<td>CL 735</td>
<td>10.9</td>
<td>5.6</td>
<td>Oct 06</td>
<td>Feb 08 (³)</td>
<td>Feb 09</td>
<td>1.4</td>
<td>0.8</td>
</tr>
<tr>
<td>CL 835</td>
<td>11.2</td>
<td>5.1</td>
<td>Sep 06</td>
<td>Mar 08</td>
<td>Sep 09 (³)</td>
<td>2.0</td>
<td>0.8</td>
</tr>
<tr>
<td>CL 1+000</td>
<td>10.1</td>
<td>4.4</td>
<td>Sep 06</td>
<td>May 07</td>
<td>Oct 08</td>
<td>1.5</td>
<td>1.1</td>
</tr>
</tbody>
</table>

Notes:
1) For all locations PVD is installed at 1.3 m c/c spacing on a triangular grid layout.
2) Degree of consolidation is estimated at 95% (unless otherwise noted).
3) At both Toll Plazas Ch 7+400 & CL Ch 735 plus Clonmacken Link WD Ch 380 additional surcharge was placed to speed the drainage and permit early release at 85% to 90% consolidation.
4) Surcharge maintained for an additional 4 months due to local rise in pore pressures unrelated to embankment loading after May 2009.
### Table 3 Summary of Basal Geosynthetic Reinforcement Performance

<table>
<thead>
<tr>
<th>Chainage (m)</th>
<th>Alluvium Depth (m)</th>
<th>Max. Embankment Height incl. Surcharge (m)</th>
<th>Geosynthetic Type &amp; Grade</th>
<th>Fill Height at initial strain &gt; 0.1% (m)</th>
<th>Max. Tensile Strain at Embankment Crest (%)</th>
<th>Max. Tensile Strain at Embankment Mid - slope (%)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>6+920</td>
<td>9.7</td>
<td>6.9</td>
<td>Basetex 400/50</td>
<td>3.6</td>
<td>1.4</td>
<td>0.6</td>
<td></td>
</tr>
<tr>
<td>8+040</td>
<td>12.0</td>
<td>9.0</td>
<td>2 layers Basetex 600/50</td>
<td>3.5</td>
<td>1.8</td>
<td>1.6</td>
<td>1.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Upper layer G1 – G4 Lower Layer G5 – G8</td>
</tr>
<tr>
<td>8+300</td>
<td>9.0</td>
<td>7.4</td>
<td>Basetex 600/50</td>
<td>0 to 1.5</td>
<td>3.4</td>
<td>2.6</td>
<td></td>
</tr>
<tr>
<td>8+700</td>
<td>8.3</td>
<td>5.9</td>
<td>Basetex 600/50</td>
<td>0 to 2.7</td>
<td>1.5</td>
<td>1.1</td>
<td>Granular fill permitted as surcharge</td>
</tr>
<tr>
<td>CL WD 380</td>
<td>4.0</td>
<td>11.1</td>
<td>Basetex 400/50</td>
<td>0</td>
<td>2.2</td>
<td>N. A.</td>
<td></td>
</tr>
<tr>
<td>CL 220</td>
<td>9.2</td>
<td>7.7</td>
<td>Basetex 400/50</td>
<td>0</td>
<td>2.6</td>
<td>2.3</td>
<td>Close to embankment failure at Ch 140 – 190m</td>
</tr>
</tbody>
</table>

Notes: 1) Single layer of geosynthetic laid transverse to embankment unless otherwise noted.

2) Maximum tensile strain is calculated as change from initial baseline readings prior to filling.
Figure 1 Site Location Plan for Limerick Tunnel PPP Project
Figure 2 Moisture content, bulk density, specific gravity and organic content of Alluvium samples from supplemental ground investigation laboratory testing programme

Figure 3 Liquid limit, plasticity index and liquidity index of Alluvium samples from supplemental ground investigation laboratory testing programme
Figure 4 Undrained Shear strength v depth derived from CPT tests and compared to UU triaxial results and corrected in-situ vane tests from adjacent Boreholes
Figure 5 Ratio of Undrained Shear Strength ($c_u$) to Effective Overburden Stress ($p_o'$)

![Figure 5](image-url)

**Figure 6** Virgin Compression Ratio ($C_c / (1+e_0)$) v Moisture Content.

![Figure 6](image-url)
Figure 7 Rate of Consolidation $C_v$ v Mean Vertical Stress

Consolidation Rate derived from Oedometer Test Results

Figure 8 Secondary Compression Ratio $C_\alpha$ v Moisture Content

Empirical Relationship ($C_\alpha = 0.00018 \times \text{mc}$) (Simons 1974)

Detailed GI Test Data

C alpha from Supplementary GI Test Data

Trendline for All LSRR GI Data
Figure 9 Location Plan of Case History Projects
Figure 10 Summary Plan of Earthworks Design Solutions
Figure 11 Detail of Prefabricated Vertical Drain (PVD) Layout

Detail 208: Fill Zones & Drainage Detail CH 0+000 to 0+080 & CH 0+320 to 1+280, Clonmacken Link Road

Detail 301: Ground Improvement with Vertical Wick Drains
Figure 12 Effect of Surcharge on Secondary Compression Ladd (1989)

Figure 13 Reduction in Rate of Secondary Compression due to Surcharge for Cohesive Soils Ng (1998)
Figure 14 $C'_\alpha/C_\alpha$ vs. AAOS % for Shannon Estuary Alluvium Conroy et al (2010)

Figure 16 Definition of Embankment Deformation Ratio ($\Delta Y/\Delta S$) Jardine (2006)
Figure 15 Typical Bridge Approach Transition Details
Figure 17 Typical Instrumentation Layout

DETAIL 402: STANDARD INSTRUMENTATION CROSS SECTION
Figure 18 Location Plan for Major Quarries
Figure 19 Bunlicken Lake Causeway Construction Sequence Plan
Figure 20 Bunlicky Lake Causeway Typical Cross Section

Figure 20: Bunlicky Lake Causeway Typical Cross Section

Detail 201: Embankment Construction at Bunlicky Lake

Detail 202: Construction at Bunlicky Lake CH 5+000m (Section 2)
Figure 22(a) Ch 4+150m Pore Pressure & Fill Height v Time

Figure 22(b) Ch 4+185m Settlement & Fill Height v Time
Figure 22(c) Ch 4+185m Deformation Ratio & Fill Height v Time

Fill Rate = 1.3m/wk

Figure 22(d) Ch 4+185m Inclinometer Deformation v Depth
Figure 23(a) Ch 8+300m Pore Pressure & Fill Height v Time

Figure 23(b) Ch 8+300m Settlement & Fill Height v Time
Figure 23(c) Ch 8+300m Deformation Ratio & Fill Height v Time

Figure 23(d) Ch 8+300m Inclinometer Deformation v Depth
Figure 24(a) Measured & Predicted Settlements v Chainage, South of Shannon

Figure 24(b) Measured & Predicted Settlements v Chainage, North of Shannon
Figure 24(c) Measured & Predicted Settlements v Chainage, Clonmacken Link

Figure 25(a) Peak Deformation Ratio v Chainage, South of Shannon
Figure 25(b) Peak Deformation Ratio v Chainage, North of Shannon

Figure 25(c) Peak Deformation Ratio v Chainage, Clonmacken Link
Figure 26(a) Ch 150m Pre-Failure Settlement & Fill Height v Time

Figure 26(b) Ch 150m Pre-Failure Deformation Ratio & Fill Height v Time
Figure 27 Clonmacken Link Ch 220m Geosynthetic Strain & Fill Height v Time