Deformation Performance and Stability Control of Multi-stage Embankments in Ireland

Performance en déformation et contrôle de stabilité de remblais construits par étapes en Irlande

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ABSTRACT: The Limerick Tunnel project includes approximately 10 km of approach roads, most of which are constructed on embankments up to 10m high within the River Shannon flood plain. The ground conditions consist of very soft organic silts up to 13m deep. A combination of vertical drains and basal reinforcement plus 2 to 2.5m of surcharge was generally adopted as ground improvement. The embankments were carefully built at controlled rates in multiple stages with continuous monitoring of performance by means of piezometers, inclinometers, settlement plates and survey monuments. The paper describes the observational approach used to control embankment stability primarily by means of monitoring filling rates, pore pressures and deformation ratio of lateral toe displacement to vertical crest settlement. Performance data relating to a failure of a 3m high embankment during the first stage of loading is included. Comparison of critical filling rates prior to minimum stability as indicated by maximum deformation ratio is presented for the Limerick Tunnel project as well as other case histories of multi-stage embankments in Ireland and some critical conclusions are drawn.

RÉSUMÉ : Le projet du Tunnel de Limerick comprend environ 10 km de voies d'accès, la plupart desquelles sont construites sur des remblais jusqu'à 10 m de hauteur dans la plaine d'inondation de la Rivière Shannon. Les conditions de sols consistent en une vase organique très molle qui va jusqu'à 13 m de profondeur. Une solution combinant des drains verticaux avec un renforcement de la base du remblai, plus une surcharge de 2 à 2.5 m de hauteur a généralement été utilisée pour améliorer le sol. Les remblais étaient soigneusement construits en plusieurs étapes avec une surveillance continuelle du comportement par des piézomètres, des inclinomètres, des plaques de règlements et des repères topographiques. Le document décrit la méthode observationnelle utilisée pour contrôler la stabilité du remblai, principalement par la surveillance des vitesses de remblaiement, des pressions interstitielles, et du ratio de déformation latérale en pied de remblai rapporté au tassement en crête. Les données du comportement d'une rupture d'un remblai de 3m de hauteur lors d'une première étape de chargement sont incluses. La comparaison des vitesses de remblaiement critiques juste avant d'atteindre une stabilité minimum telle qu'indiquée par le ratio de déformation maximale est présentée pour le Tunnel de Limerick, ainsi que pour d'autres études de cas de remblais construits par étapes en Irlande, et quelques conclusions importantes sont données.

KEYWORDS: Ground Improvement; Surcharge; PVD; Multi-stage Embankments; Performance Monitoring; Observational Design.

1 INTRODUCTION.

1.1 *Project Description*

The Limerick Tunnel project is located to the south and west of Limerick City in SW Ireland and provides a dual carriageway bypass of the city via an immersed tube tunnel beneath the River Shannon. The route passes through flat, low lying alluvial flood plains of the River Shannon and its tributaries. 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. The flood plain of the River Shannon is underlain by extensive deposits of very soft to soft alluvium comprising mainly organic silt to depths typically from 3 to 13m deep.

The embankments were carefully built at controlled rates in multiple stages with continuous monitoring of performance by means of piezometers, inclinometers, settlement plates and survey monuments. This paper describes the observational approach used to control embankment stability primarily by means of monitoring filling rates, pore pressures and deformation ratio of lateral toe displacement to vertical crest settlement. Principle methods adopted for earthworks along the project include one of more of the following ground improvement solutions:

- Full or partial excavation and replacement;
- Prefabricated Vertical Drainage (PVD);
- Geosynthetic Basal Reinforcement;
- Multi-Stage Construction Techniques; and
- Surcharging.

A more detailed description of the design, construction and performance of embankments along the project is contained in Buggy & Curran (2011).

1.2 *Site Characterization and Alluvium Properties*

A brief summary of the engineering properties of the soft alluvium follows but a much more extensive description is given in Buggy & Peters (2007). 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 lightly overconsolidated 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 for the alluvium are summarized as follows:

- Natural moisture content - 40 to 120 % in organic silt and 150 to 300 % in peaty layers;
- Liquid Limit - 40 to 150 % in organic silt and 150 to 300 % in peaty layers;
- Plasticity Index – 30 to 75 %;
- Organic content (loss on ignition) typically 2 to 10 % but up to 34% in peaty layers;
- Undrained strength ratio c_u / p_o' varies 0.36 CAUC triaxial tests; 0.3 DSS test; 0.2 CAUE triaxial tests; 0.3 average assumed in design; and
- Coefficient of consolidation C_v = 0.5 to 4 m^2/yr (lab tests) and 0.8 to 1.5 m^2/yr (field back calculated).

The alluvium sediments are underlain by deposits of predominantly fine grained glacial tills with occasional coarse grained layers and limestone bedrock.

2 MULTI-STAGE EMBANKMENT STABILITY

At Limerick Tunnel the designers adopted the undrained strength analysis approach as developed by Ladd (1991). This employs a normalised undrained strength ratio c_u / p_0' to predict the operational shear strengths 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.

A cautious design value of C_{vr} = 1 m^2/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 spacing but in one 200m long high fill area drains were installed at 1.0 m c/c spacings. 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 of the 6 km total embankment length requiring PVD and surcharge, typically where the total temporary embankment height (including surcharge fill) exceeded 6m.

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) 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:

- Large ground movements due to volume changes can occur as instability is approached;
- 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 ΔY to maximum settlement at the crest ΔS (see Figure 1). The limit criteria for such ratios will be different for single stage compared to multi-stage embankments and indeed will vary with each site due to soil material properties, embankment geometry, soil profile and loading rate;
- Similarly as instability is approached the ratio of pore pressure change in the foundation soils to increased total loading approaches and exceeds unity; and
- 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.

Deformation ratio limits reported in CIRIA C185 (1999) typically range from 0.3 to 0.4 for embankments on soft ground. Data from Japan published by Wakita & Matsuo (1994) has suggested that the deformation ratio for a given degree of stability reduces as total settlement increases, failures being expected for deformation ratios in excess of 0.4 for observed settlements greater than 2m.

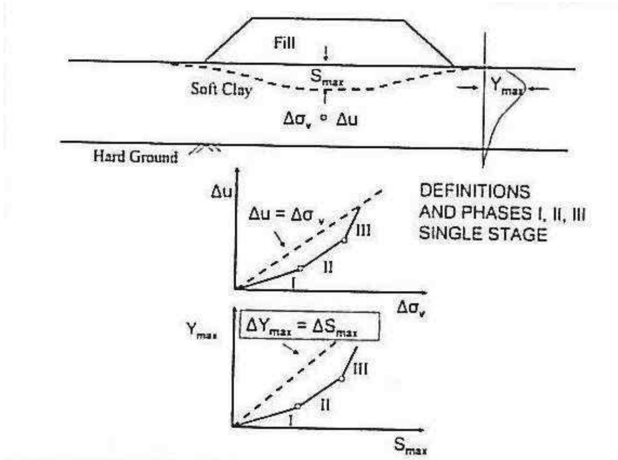


Figure 1 Definition of Embankment Deformation Ratio ($\Delta Y/\Delta S$) (Jardine, 2002)

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 $\Delta u/\Delta\sigma_v > 1.0$;
- 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; and
- Incremental change in settlement or toe movement > 0.1m between consecutive readings.

3 INSTRUMENTATION MONITORING

A total of 13 fully instrumented cross sections were selected at representative locations and near structures where temporary fill heights were greatest. A standard instrumentation cross section included a pair of settlement plates 5m 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. Inclinometers extended to stiff glacial till soils or bedrock were installed at the embankment toes.

Settlement plates and toe survey monuments were generally arranged in pairs at 50 m c/c spacing along the mainline. Active areas of filling with settlement rates > 20 mm / week required twice weekly monitoring but daily monitoring was triggered when monitoring threshold limit values were exceeded.

4 EMBANKMENT PERFORMANCE

4.1 *Deformation Ratio & Stability*

A typical filling rate and deformation ratio history for the instrumented location at Ch 4+185 m is shown on Figure 2. During initial filling to heights of 4 m the deformation ratio rapidly rose to local maximum values of 0.4. The ratio then reduced to below 0.2 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 along the project..

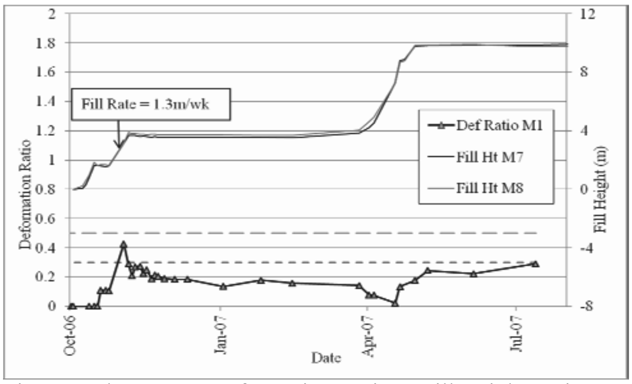


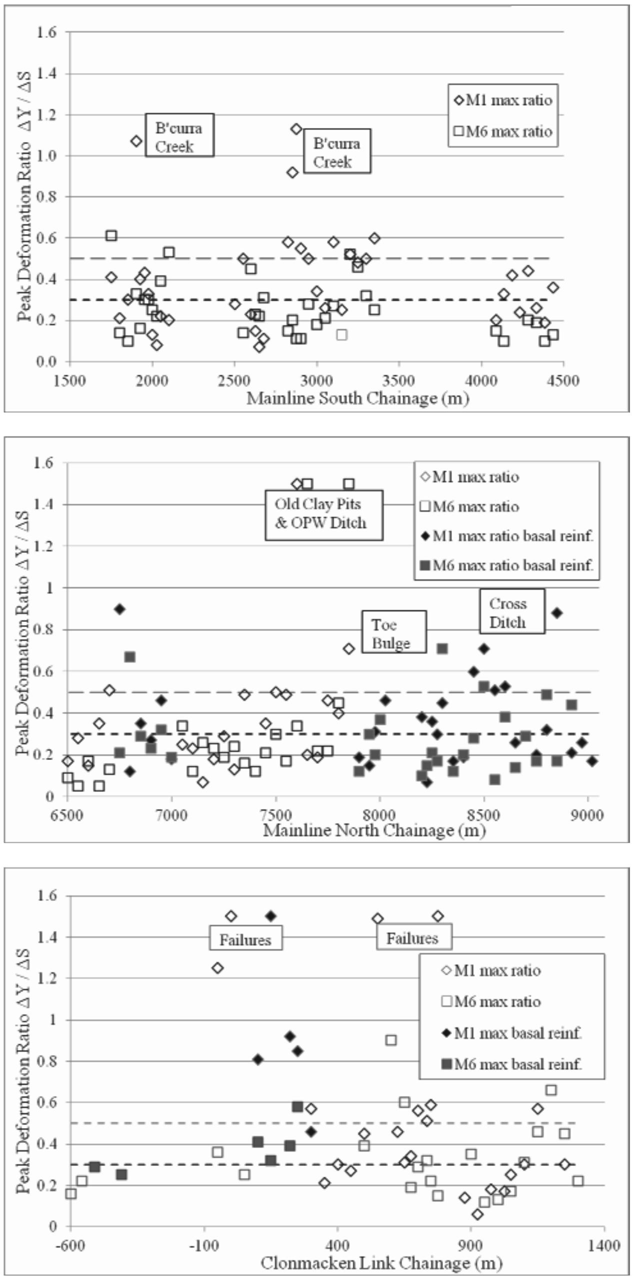
Figure 2 Ch 4+185m Deformation Ratio & Fill Height v Time

The distribution of measured peak deformation ratio throughout areas of the project improved by PVD and surcharge is shown in Figures 3 (a), (b) & (c) for the mainline embankment south and north of the River Shannon plus Clonmacken Link respectively.

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 ratios 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. Around 90% of all survey monitored sections demonstrated a deformation ratio of under 0.6.



Figures 3 (a), (b) & (c). Peak Deformation Ratio v Chainage, Mainline South, North & Clonmacken Link respectively.

4.2 *Embankment Failures at 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 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); and
- High filling rates around 1.5 to 3 m/week.

Typical deformation ratio history for instrumented location at Ch 0+150 m is shown on Figure 4. 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 resulted in failure.

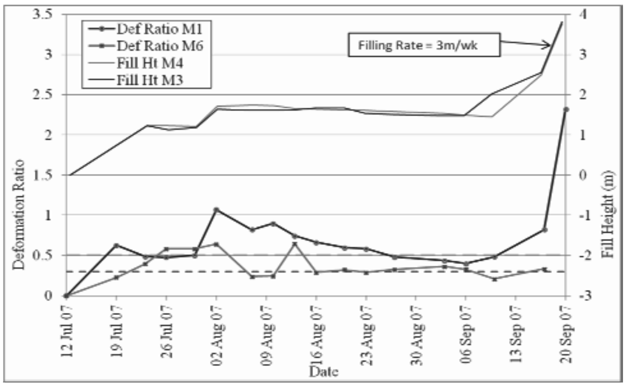


Figure 4 Ch 150m Deformation Ratio & Fill Height v Time

5 PEAK DEFORMATION RATIO & FILLING RATES

The variation of peak deformation ratio associated with the maximum filling rate has been investigated for the Limerick Tunnel project as well as other published case histories of multi-stage embankments in Ireland. The maximum filling rate is defined herein as the local filling rate typically measured over a period of a few weeks immediately prior to the peak deformation ratio being observed. This is not the same as the slower average filling rate for a discrete stage in construction.

The data is presented in Figure 5 and includes 18 data points from Limerick Tunnel representative of 6 km length of embankment on PVD improved ground and a range of depths of soft ground from 3 to 11m. Also included are 9 data points from 4 sites in Cork, Bunratty Co. Clare, Derry and Athlone with a diverse range of embankment geometry from 2:1 to 3:1 side slopes (Derry & Athlone with toe stability berms); depth of soft ground from 6 to 13m; range of C_v typically from 0.5 to 3 m^2/yr and PVD spacing 0.6 to 1.4 m (Dunkettle Cork did not contain PVD and exhibited a range of C_v from 7 to 16 m^2/yr).

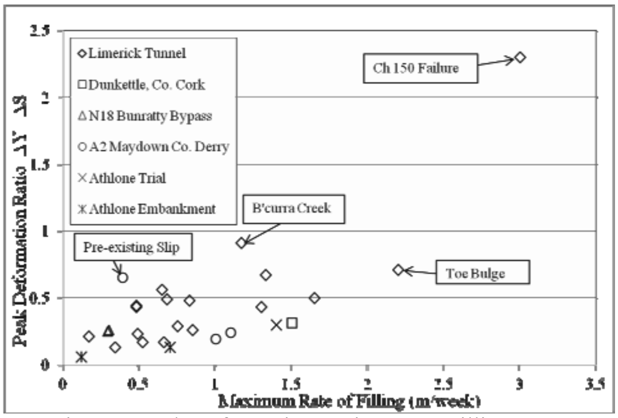


Figure 5 Peak Deformation Ratio v Max Filling Rate

While there is considerable scatter in the data, there is a discernable trend of increasing deformation ratio and thus decreasing stability as the local filling rate increases. Below maximum filling rates of 0.5 and 1m / week, deformation ratios generally are below 0.25 and 0.5 respectively, suggesting stable conditions. An exception to this occurs at A2 Maydown site in Derry where significant deformation ratio of 0.65 occurred during construction at a modest maximum filling rate of only 0.4 m / week. This was found to be caused by a pre existing failure plane and remedied by addition of toe stability berms. At Limerick Tunnel filling rates in excess of 2 m / week were consistently associated with large observed deformations or failures. Local failures were strongly influenced by the presence of creeks, ditches or excavations and occurred at filling rates above 1m / week. Peak ratios were nearly always observed during first stage filling at embankment heights of 4m or less. Only 4 peak ratios (15% of the total for all sites) occured at embankment heights over 5m during second stage or near final embankment height.

6 CONCLUSION

Deformation ratios offer a reliable method for controlling stability of multi-stage embankments when used in conjunction with pore pressure instrumentation. An assessment of critical deformation ratio values during loading should be based on modelling of the specific embankment geometry and soft ground properties, trial embankments and case history precedence in similar conditions. For the specific conditions at Limerick Tunnel a limit deformation ratio of 0.6 was shown to give satisfactory performance and acceptable stability.

Excessive lateral deformation related to local failure occurred at several locations in the vicinity of creeks, ditches and hisotrical excavations located within 10m of the embankment toe. More general failure of embankments occurred at 2 locations during construction, in both cases related in part to excessive filling rates. Local filling rates in excess of 1 m / week have a significantly higher risk of failure and rates below 0.5m / week are advisable to sustain a well controlled, stable stage filling for typical Irish soils.

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8 REFERENCES

Buggy, F. J. & Peters, M., 2007. Site Investigation and Characterisation of Soft Alluvium for Limerick Southern Ring Road - Phase II, *EI Proc. Conf. Soft Ground*, Port Laoise, paper 1.6.

Buggy, F. J. and Curran, E., 2011. Limerick Tunnel Approach Roads – Design Construction & Performance. *Geotechnical Society of Ireland Meeting, Engineers Ireland,* Dublin 8th December, 2011.

CIRIA Report C185, 1999. Observational Method in Ground Engineering. *CIRIA* London.

Connolly, C., Davitt, S. & Farrell, E. 1990. The Design and Construction of Bunratty Bypass, *Engineers Ireland Meeting* Limerick.15th October 1990.

Creed, M. J. 1996. Dunkettle Embankments, Glanmire Co. Cork – Pore Water Pressures Reviewed. *Engineers Ireland Conference Road Embankments on Soft ground in Ireland.* Dublin.

Dauncey, P.C., O'Riordan N.J. & Higgins J. 1987. Controlled failure and back analysis of a trial embankment at Athlone. *Proc. IX Europ. Conf. Soil Mechnanics.* Dublin pp 21 – 24.

Jardine, R.J. 2002. Stability and Instability: soft clay embankment foundations and offshore continental slopes." Keynote Paper. *International Symposium on Coastal Geotechnical Engineering In Practice.* Volume 2, Balkema, Rotterdam. pp 99-118.

Kamrat-Pietraszewska, D., Buggy, F., Leoni, M. & Karstunnen, M. 2008. Numerical Modelling of an Embankment on PVD-Improved soft Alluvium – Limerick Southern Ring Road Phase II. *BCRI 08 Symposium* NUI Galway pp 443 – 452.

Ladd, C.C. 1991. Stability Evaluation During Staged Construction. *ASCE Journal Geotech. Engng Divn.* Vol 117. No. 4. pp 540 -615.

Long, M. & O'Riordan N., 2001. Field behavior of very soft clays at Athlone embankments. *Géotechnique* Vol. 51 (4) pp 293 – 309.

Raven, K. & Anketell-Jones, J. 2012.A2 Maydown to City of Derry Airport – Construction over Soft Ground. *Proc. Conf. Geotechnics on Irish Roads 2000 - 2010*, Portlaoise. 11 October 2012.

Wakita, E. & Matsuo, M. 1994. Observational design method for earth structures constructed on soft ground. *Géotechnique* Vol. 44 (4) pp 747 – 755.