EMBANKMENTS FOR THE LIMERICK TUNNEL APPROACH ROADS: Case narrative

Highway embankments in installments (EN) – Αυτοκινητόδρομος σε δόσεις (EL)

Note: In the 3-part description that follows, actual findings from geotechnical investigations and reports are embedded in a case narrative developed for educational purposes; to this end, the narrative involves fictitious characters and some hypothesized preliminary calculations. The description was developed primarily on the basis of the project description given in Buggy and Curran (2011), and includes some supplementary information specific to the cross section to be analyzed (see Figure 1) from the project's design report (Alliance, 2006).

A highway project, which includes the submerged tunnel crossing of the River Shannon south of Limerick, Ireland, necessitated the construction of several kilometers of embankments, typically 3 to 8 m high. The embankments were to be constructed on soft alluvial deposits (i.e. deposited by river water), consisting mainly of organic silt/clay; firm material (glacial till and limestone) is found below a depth which, in some places, is up to 13m. Existing local experience indicated that embankments would have problems if constructed on such soft materials.

<u>PART A – Why is soil improvement needed?</u>

After the penultimate year of her civil engineering studies, Cara is awarded a summer internship with the consulting company performing the geotechnical analysis for the Limerick Tunnel approach roads. Her supervisor, Ms Moran, is a congenial senior civil engineer who enjoys sharing her experience with current and future colleagues. She prefers Cara to be convinced for herself that it would not be a good idea to construct the embankments without implementing some soil improvement measures. As a first assignment, she gives Cara one of the representative cross sections with a shallow soft organic silt/clay alluvial layer, 3m thick, which is shown in Figure 1, and asks her to "check it out".

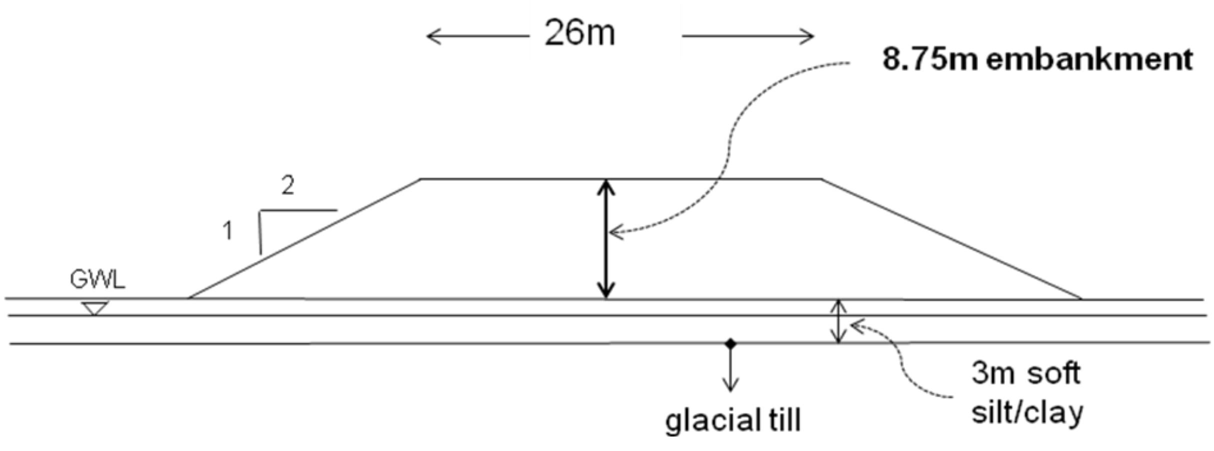

Figure 1. A simplified version of cross section at Chainage 4+150 showing required embankment height. Ground water level (GWL) is at -1m.

Ms Moran suggests working through the assignment in two steps. First thinking the problem over and then, after a discussion between the two of them, performing the calculations. She further explains that "thinking the problem over" includes the following tasks:

(I) Identifying the different things that can go wrong or, equivalently, the different modes of failure or of unsatisfactory performance,
(II) Deciding on methods of analysis for each mode of failure, and
(III) Trying to select suitable soil parameters for these analyses.

Cara has access to some site-specific data and results of the geotechnical investigations from other similar local projects reported in Table 1 of the article by Buggy & Curran (2011), as well as to geotechnical engineering textbooks (an excerpt from Table 1 is reproduced herein at the end of the narrative as Table A1). Being happy that her supervisor is willing to spend extra time teaching her, Cara decides to surprise Ms Moran with doing as many calculations as she can manage on her own before their discussion. Even if she lacks some data, she will go ahead by making plausible estimates.

Cara is most apprehensive about Task (III), but she decides to worry about that after she thinks about Tasks (I) and (II); besides, Ms Moran only asked her to *try* to do Task (III). She starts by making a list of the bad things that can happen. She decides to include every possibility, even improbable ones, and omit later any that are irrelevant to the situation. The list includes:

(Ia) Excessive settlement,
(Ib) Bearing capacity failure,
(Ic) Instability of the embankment slope, and
(Id) Slope instability involving both the embankment material and the foundation soil.

Cara makes a note to discuss any concerns about her list with her supervisor.

For the settlement of the silty/clayey material, she plans to calculate the primary consolidation settlement, although she is not sure whether to use the equation with the coefficient for volume change (m_v) or the equation with the compression index (C_c) (to check the worst case scenario, she will do them both and see how the results look…). In each case, she needs the unit weight of the two soils and she finds an average value for the alluvium of 16 kN/m^3 in Buggy & Curran (2011). For the embankment, she assumes that a value of 20 kN/m^3 is reasonable for a well-compacted material. She will also perform a calculation for the time necessary for the consolidation settlement to be completed, and for this calculation she needs the coefficient of consolidation, c_v.

With regard to bearing capacity failure, she decides that she may not need to worry about this, considering the significant width of the embankment relative to the small thickness of the foundation material, which does not leave sufficient space for a bearing failure mechanism to develop beneath the embankment. She reasons that, since the geometry resembles a one-dimensional loading situation, it is difficult for the soil to move laterally, hence the full 2-D shear deformation involved in a bearing capacity failure is not of concern in this problem.

For the slope stability calculations, she needs the shear strength parameters of the two soils. Guessing that the soft organic soils will tend to compress during shear, she anticipates that the short-term stability, i.e. under undrained conditions, is of major concern, because the pore pressure will tend to increase upon loading. With time, as the excess pore water pressures dissipate, the effective stress will increase and so will the shear strength, but by then the soil may have failed! Since she has some values for the undrained shear strength, c_u, of the foundation material as a function of the vertical effective stress, p_o', she decides to assume some values for the embankment and to perform the stability calculations as well before she meets with Ms Moran. She finds an example of a highway embankment design in a textbook on the Internet and uses the effective shear strength parameters for the embankment material from this example, which are $c'=25$ kN/m^2 and $\phi'=20°$; she realizes that these values are very much dependent on the type of soil to be used, but she hopes that their combination corresponds to a soil acceptable for embankment construction. In any case, because she felt more comfortable with the choice of the unit weight for the embankment soil than with the choice of the shear strength parameters, she makes a note to ask Ms Moran how she would think about making such an estimate.

[It is recommended that the assumptions and calculations of Part A be discussed before proceeding with Part B.]

PART B – The logic behind soil improvement measures & respective calculations

Ms Moran discusses with Cara the proposed soil improvements for the soft soils, which include full or partial excavation and replacement with suitable backfill material, accelerating consolidation drainage using prefabricated vertical drains (PVD), geosynthetic basal reinforcement, multi-stage construction and surcharge. Excavation is not an attractive option, due to the combined cost of temporary stabilization works, imported backfill and disposal of excavated unsuitable material. Hence, soil deposits deeper than 4m are not be excavated and even for shallower deposits, such as the 3-m deep alluvium layer in Figure 1, soil improvement measures are preferred. Ms Moran would like Cara to help with the analysis for the combined application of PVD, surcharge and multi-stage construction, so she describes to her the general concept and the main steps of the analysis, building on the calculations already performed by Cara.

As a start, Cara considers again the cross section in Figure 1, only this time she will use the soil parameters determined specifically for the existing soils in the vicinity of the cross section and for the embankment material, which are included in Table 1. Ms Moran explains that the low shear strength of the alluvium will be improved by allowing it to consolidate under increasing load. This is achieved by constructing the earthworks in stages with successive layers, and holding each stage load constant until the pore water pressure measurements in the field confirm a significant decrease in the excess pore water pressure. The role of the vertical drains is to help reduce the consolidation time by decreasing the lengths of the drainage paths. The thickness of the first fill layer is equal to the maximum embankment height the alluvium can withstand with its undrained shear strength in its natural state. Each loading cycle is followed by consolidation, resulting in increased vertical effective stress and, hence, increased undrained shear strength, as described by the relationship $c_u=0.3p_o'$ for normally consolidated soil, where p_o' is the vertical effective stress; the validity of this relationship has been confirmed for the alluvium below a slightly overconsolidated layer close to the surface. Hence, an increasingly higher undrained shear strength can be used in the slope stability calculation to determine the new embankment height the soil can sustain at each loading stage. The process is repeated until the maximum embankment height, with the surcharge, is attained.

Table 1. Site-specific parameters values from the design report (Alliance, 2006) or reported by Buggy and Curran (2011) (B&C 2011).

Parameter	Source of the parameter	Design value
	Alluvium	
Unit weight, γ_a	Design report	17 kN/m^3
Moisture content, w%	Design report, cross section average	100%
Specific gravity, G_s	B&C (2011), Figure 2, average	2.63
Void ratio, e_o (calculated assuming 100% saturation from γ_a and G_s)		1.23
Compression index, C_c	B&C (2011), Figure 6	$[C_c /(1+ e_o)]= 0.33$ (for w=100%)
Coefficient of consolidation, c_v	Design report	1 m^2/yr
Coefficient of consolidation, c_h	B&C (2011), page 4	1 m^2/yr
Undrained shear strength c_u	Design report (depth-weighted average for the cross section)	25 kN/m^2
Angle of shearing resistance in terms of effective stress, ϕ’	B&C (2011), page 3	28°
Cohesion intercept in terms of effective stress, c’	Not mentioned in the design report, apparently c’=0	0
	Fill	
Unit weight, γ_f	Design report	21 kN/m^3
Undrained shear strength c_u	B&C (2011), page 7	75 kN/m^2
Angle of shearing resistance in terms of effective stress, ϕ’	Design report	35°
Cohesion intercept in terms of effective stress, c’	Not mentioned in the design report, apparently c’=0	0

The required amount of surcharge is calculated on the basis of the desired reduction in secondary compression. Cara is surprised that, just as in the case of primary consolidation, it is also possible to get rid of some secondary compression with a surcharge. Ms Moran reminds Cara that they are calculated separately because they are due to different mechanisms (primary consolidation being due to squeezing out of water and secondary compression being due to particle rearrangement). However, in reality the two proceed simultaneously while excess pore pressures dissipate and, hence, the surcharge not only squeezes out some excess water, but also causes some particle rearrangement as well.

After giving Cara a general idea of the design strategy, Ms Moran proceeds with describing the main features of each calculation step and the relevant decisions that have already been made. The calculation steps are as follows.

<u>Step 1</u>. Choose a drain spacing to give a reasonable period to achieve the complete primary consolidation on the basis of construction scheduling requirements (for this project < 2yr).

A triangular pattern is chosen for the installation of the prefabricated drains, with a center-to-center spacing of 1.3m. The dimensions of the specific PVD selected are 10cm by 3mm. With this information, Ms Moran asks Cara to confirm that the 1.3m spacing meets the requirement that primary consolidation will be completed in less than 2 years.

Step 2. Determine the additional surcharge height, h_s, needed to reduce the secondary compression to within a range of 20–50mm.

The reduction in secondary compression is estimated using a correlation between the ratio of the coefficients of secondary compression with (C_α') and without (C_α) surcharge and the Adjusted Amount of Surcharge (AAOS), defined as:

$$AAOS = (\sigma_s' - \sigma_f')/\sigma'_f \text{ (expressed as percentage)} \qquad (1)$$

where σ_s' is the maximum vertical effective stress experienced by the soil during the hold period for the surcharge and σ_f' is the final vertical effective stress after surcharge removal. The correlation used between C_α'/C_α and log(AAOS) is given by the straight line relationship in Figure 13 by Buggy and Curran (2011) can be used to determine σ_s' and hence h_s; this empirical relationship can be expressed as:

$$C_\alpha'/C_\alpha = 1.85 - 1.08 \times \log(AAOS) \qquad (2)$$

Step 3. Evaluate slope stability for the different stages of construction (to simplify the description, a two-stage construction is assumed).

Step 3a. Calculate the maximum initial embankment height, say h_1, that corresponds to a stable slope for the undrained strength of the alluvium in its natural state, i.e. prior to any loading.

Step 3b. Calculate the degree of consolidation for different hold times under the load from the embankment height h_1; calculate the increased vertical effective stress p_o' and hence calculate the new $c_u = 0.3p_o'$. For the overconsolidated soil close to the surface it is possible that the increased p_o' is smaller than the preconsolidation pressure for that soil, in which case no change to the initial c_u is made.

Step 3c. Perform slope stability analyses for the maximum embankment height needed, h_2 (i.e. h_2 = required height for highway embankment plus the additional surcharge height to be later removed), and determine the required c_u for the embankment slopes to be stable. This c_u value determines the necessary hold time, t_{h1}, for Stage 1. Ms Moran notes that Step 3c can be completed before Step 3b and, from the required c_u value, the degree of consolidation and necessary Stage 1 hold time can be found. However, as an educational exercise, she recommends Cara to consider a few pairs of $t_{h1} - c_u$ values in Step 3b.

Step 3d. Where it was found that too much time was needed to complete the embankment construction, including placing and removing the surcharge, then a geosynthetic basal reinforcement was used, hence increasing the stability and allowing thicker layers to be constructed at each stage. Note: this was the case for cross sections with deeper alluvium layers (e.g. 8 m).

Step 4. Perform a long-term slope stability analysis with the effective stress shear strength parameters.

PART C – Instrumentation of embankments during construction

Monitoring included settlement plates, piezometers (to measure pore pressures) and inclinometers (to measure lateral movements). Filling schedules and hold times were altered as necessary to be consistent with the observed behavior. Apart from using the data from settlement plates and piezometers to confirm that consolidation proceeds as predicted, the embankments were also monitored for lateral movements, which, if large, are a sign of impeding instability. For this purpose, the ratio of the lateral movement at the toe of the embankment, ΔY, to the maximum settlement at the crest, ΔS, was recorded during construction. The threshold limits for the observed quantities, including the ratio $\Delta Y/\Delta S$, were determined using finite element analyses as part of the design. Consideration of these threshold limits imposes a further restriction on the maximum stable embankment heights calculated in Step 3 as described in Part B.

References

Alliance [2006]. i.e. Roughan & Donovan – FaberMaunsell Alliance [2006], Limerick Southern Ring Road Phase II, Limerick Tunnel PPP Scheme, Geotechnical Interpretative and Earthworks Design Report, Volume 2: Part 6 – Dock Road Interchange to Bunlicky Lake (Ch 3+780 to 4+460m), January 2006.

Buggy, F. and E. Curran [2011]. "Limerick Tunnel Approach Roads – Design, Construction and Performance", Paper presented to Engineers Ireland, Geotechnical Society of Ireland, Dec. 8, http://www.engineersireland.ie/EngineersIreland/media/SiteMedia/groups/societies/geotechnical/Limerick_Tunnel_Approach_Roads_Nov_2011.pdf?ext=.pdf (accessed September 21, 2012).

Table A1. Comparative Data for Road Embankments on Soft Alluvial / Organic Soils in West Ireland (excerpt from Buggy and Curran, 2011).

Project Location	**N18 Bunratty Bypass Overbridge West of Ratty River** *Farrell, Davitt & Connolly (1996)*	**N18 Bunratty Bypass Ratty River Bridge, Eastern Approach** *Farrell, Davitt & Connolly (1996)*	**Athlone Bypass Trial Embankment** *N. J. O'Riordan (1996)*	**Limerick Southern Ring Road Phase 2** *F. Buggy & M. Peters (2007) & current paper*	**Galway Eastern Approach Road** *Flood & Eising (1987)*	**North Approach Rd Mallow St Bridge. Limerick** *R. Galbraith (1996)*
Soil Description	Soft organic silt w/ peat layers up to 1m thick. Surficial firm crust up to 1m thick.	Soft alluvial soils	Soft grey organic clay	Soft organic estuarine silt	Organic silty clay Peat Calcareous Marl	Soft organic silt
Max Thickness of Soft Soils (m)	7m	12m	15m	13m (peat layers up to 2.5m)	10m	8m
Natural Moisture (%)	30 – 100 in organic silt 100 – 500 in peat	40 - 70	40 - 80	40 – 120 in organic silt 150 – 300 in peat	100	52 - 64
Liquid Limit	Similar to natural moisture	-	60	40 – 150 in organic silt 150 – 300 peat	-	47 - 56
Plasticity Index	-	-	40	30 - 75	-	21 - 23
Organic Content (%)	42% typical	-	-	0 – 20 in organic silt 20 – 50 in peat	-	
Undrained Strength Ratio Cu / Po'	0.3 assumed in design 0.28-0.42 back calculated from field vane test data.	0.5 – 0.6 lab measured 0.25 assumed	0.25 – 0.30	0.30 design 0.20 KoCUE 0.30 DSS 0.36 KoCUC	0.29 – 0.64 0.3 design	0.25 – 0.41 0.3 design
Coefficient of Secondary Compression C alpha	0.00018 w	0.00018 w	0.016	0.00018 w	0.015 Peat 0.016 organic clay	
Coefficient of Consolidation Cv (m^2/yr)	0.35 - 0.5 m^2/yr Ch = Cv lab & back calculated from field	1 m^2/yr in critical layers Ch = Cv lab	1.5 m^2/yr back calculated from field performance	0.5 to 4.0 lab Cvh = 0.9 to 1.5 m^2/yr back calculated field	Cvh – 12 m^2/yr	Cvv = 0.4 -2.6 m^2/yr. Cvh=0.5 m^2/yr derived from standpipe tests
Coeff Vol Change Mv (m^2/MN)		1.7-0.5				0.8-1.5
Compression Index Cc		0.2 – 0.5 for w<70%	0.35 ave $Cc/(1+e_0)$	0.1 – 0.4 for w = 40 to 120%	0.4 ave $Cc/(1+e_0)$	
Max Embankment Ht (m)	9m excl. surcharge	3m	8.5m	9.5 m max 3 – 5 m typical (excl surcharge)	2m	5.5m