Case Studies Used in Instruction to Achieve Specific Learning Outcomes: The Case of the Embankments Constructed for the Approach to Limerick Tunnel, Ireland

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CASE STUDIES USED IN INSTRUCTION TO ACHIEVE SPECIFIC LEARNING OUTCOMES: THE CASE OF THE EMBANKMENTS CONSTRUCTED FOR THE APPROACH TO LIMERICK TUNNEL, IRELAND

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ABSTRACT

This paper gives an example of a case study written for instructional purposes, in order to support the achievement of specific learning outcomes which include (i) identifying modes of failure and (ii) selecting appropriate soil parameter types and values. Case writing was based primarily on information from a detailed publicly available article, supplemented with additional input from one author of this article. The case narrative is accompanied by annotated calculations, which follow the general design philosophy of the project. The case focuses on two of the main issues for the geotechnical design of the highway embankments close to the Limerick Tunnel, which are founded on very soft organic fine grained material. First, secondary compression, which is sizeable for this highway project, required surcharging to reduce the rate of long-term settlement. Second, the low undrained shear strength and high compressibility of the foundation material required construction of the embankments in stages, to achieve a degree of consolidation necessary for increased vertical effective stress, increased shear strength and reduced compressibility. This paper includes the case narrative, excerpts from the accompanying calculations, and comments on the instructional decisions involved in the preparation of both.

INTRODUCTION

The use of case studies has been a staple component in geotechnical engineering education for decades (Rogers, 2008). In contrast, the good practice of designing modules, courses and study programs on the basis of learning outcomes is relatively new. Orr (2011) started the discussion on the types of learning outcomes that can be achieved when using cases in geotechnical instruction. Orr and Pantazidou (2012) continued with proposing a list of learning outcomes best highlighted with the use of case studies. The aim of this paper is to provide an example of selecting a case study and preparing supporting material with specific learning outcomes in mind. A systematic approach to defining learning outcomes must differentiate them from general instructional purposes. General purposes may be either affective (e.g. to motivate students to study the subject matter through using case studies with a dramatic element, such as failures) or cognitive (e.g. to explain the construction issues associated with particular design decisions). In contrast to general purposes, learning outcomes state what the students will be in a position to do after successfully completing a course. Hence, for a close fit of a case study to a particular course, a good match between the course and the case study contents alone is not sufficient. In addition, the case study must support specific learning outcomes, suitable for the nature of the subject and the level of the students. This paper discusses these considerations with the aid of the decisions made during the preparation of a case study involving embankments constructed over soft alluvium for the approach to the Limerick Tunnel in Ireland.

SELECTING THE CASE STUDY

The decision to draw what type of material for a case study from a specific project partly depends on who makes it. Practitioners involved in the project will tend to favor the innovative or the challenging aspects of the case; hence the information included in an article on the project will mostly highlight these aspects. For the instructor, the major decision is whether the case study will be presented in a lecture format or whether the students will be actively involved with the case study material, evaluating data and performing calculations themselves. For the lecture format, the information required is minimal, as the students will either get a general idea of the project, or follow in detail a limited part of it. On the contrary, if the students get actively involved with the case study, they need extensive information, in order to choose from it what is relevant, but still somewhat circumscribed, so that they do not get lost in lengthy reports and appendices with data. This active involvement of students with the case material is best suited to support them in achieving learning outcomes past the lower levels of recognition and recalling, to the higher levels of application and analysis (for a detailed discussion on learning outcome levels, see Anderson et al., 2001).

From the wide variety of learning outcomes associated with a geotechnical curriculum, the authors have proposed elsewhere (Orr and Pantazidou, 2012) a list of 10 broad learning outcomes that will be reinforced by using suitable cases. These outcomes are reproduced herein in Table 1. The list is not considered to be definitive but rather meant to invite the geotechnical community to modify and augment it. Such a list can guide decisions for what material to include in a case study and which case studies to select for a particular course.

Table 1. Learning outcomes achievable from geotechnical courses listed in increasing order of performance level (adapted from Orr and Pantazidou, 2012)

No.	Definition of learning outcome Students have the ability to:
1.	Identify potential critical modes of failure
2.	Apply corresponding methods of analyses already covered in course (presupposes No 1)
3.	Select the appropriate type of soil parameter values for specific methods of analyses
4.	Assess the variability of experimental data
5.	Select appropriate calculation models for solving geotechnical problems
6.	Determine the soil profile and the specific soil parameter values to be used in geotechnical design (presupposes No 4)
7.	Choose appropriate safety elements (related to No 8, 9)
8.	Assess the complexity and uncertainties of a design situation
9.	Be aware of the professional responsibilities pertaining to geotechnical projects
10.	Consider the ethical dilemmas in geotechnical practice

WRITING THE CASE STUDY

Rather than considering a case study to be an account of an interesting project, a case study suitable for instruction may better be viewed as a story with a technical plot. Thanks to the popularity of the case study method, or case method, as a teaching technique in many disciplines, there exist guidelines on how to write cases for instruction (Herreid, 1997). Cases can be written to (a) present a decision that needs to be made, (b) guide students to focus on answering questions like "what is going on here?" or (c) present a full, finished story (Herreid, 1994). To take advantage of both the freedom allowed by an unfinished story and the interest in what happened at the end, a case can be written and given to students in parts (this is the approach adopted herein). In this way, the students can explore alternatives unbiased by the actual issues of the real project. Finally, for a better match of the actual case to the instructional purposes, as well as when wishing to avoid involving real people, writers have the option to embed the facts of a case in a fictional story (Herreid, 2002).

It is recognized that many cases are best developed from scratch (Herreid, 1994). These can be customized for instructional purposes and written for the intended audience, i.e. the students. Depending on their intended use, some cases are short, while others are many pages long with extensive data. Geotechnical engineering case studies often fall in this latter category if it is desired to get students involved with material such as maps, drawings, site data, etc. When selecting engineering cases involving analyses for use in instruction, it is most convenient for the instructor when the case includes this type of supporting material, as well as detailed analyses with calculations (like the teacher's solution manual for textbook problems). Clearly, a comprehensive case study with its accompanying material cannot be presented in a typical published paper. While the main elements of a case study envisioned for instructional use can be presented in a paper, the additional information required needs to be made available to instructors by other means. This is the solution adopted herein. The following section includes the full case narrative, while some representative excerpts from the supporting material are discussed in the respective section. The supporting material in its entirety is available at http://users.ntua.gr/mpanta/Teaching_EN/.

THE LIMERICK CASE STUDY

Whereas the case study chosen as the example in this paper is written for the students, the entire set of case study material is written for the instructors, who need to decide whether the case study is suitable for their courses, ideally on the basis of minimal prior information. To this end, the summary information in Table 2 precedes the presentation of the case itself.

Table 2. Information necessary to match a case with a course and specifics about the Limerick case study

Information type	Case specifics
Geotechnical course	Undergraduate
Geotechnical topic	Consolidation settlement, vertical drains, secondary compression, undrained strength, slope stability
Learning outcome(s)	1. Identify potential critical modes of failure 2. Apply corresponding methods of analyses already covered in course 3. Select the appropriate type of soil parameter values for specific methods of analyses

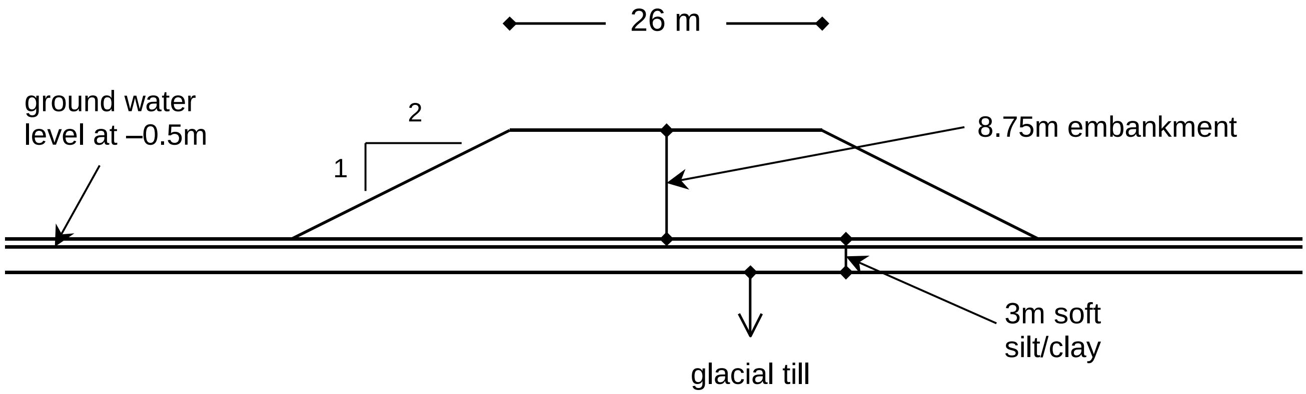

Fig. 1. A simplified version of cross section at Chainage 4+150 showing required embankment height.

Case narrative: "Highway embankments in installments"

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 Fig. 1) from the project's design report (Alliance, 2006). [Notes for the instructor are interspersed in the narrative within bold square brackets.]

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 thick. Existing local experience indicated that embankments would have problems if constructed on such soft materials.

PART A – Why is soil improvement needed?

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 Fig. 1, and asks her to "check it out".

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. 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.

[Teaching Option 1 – Students are given Part A up to this point and asked: "Suppose you are in Cara's place; how would you go about the tasks involved in "checking out" the embankment cross section?"
Teaching Option 2 – Students are given all of Part A, including the paragraphs below, which describe Cara's thinking process and decisions, and are asked: "Suppose you are Cara's co-worker and the two of you were to discuss her approach before she meets with Ms Moran; would you recommend any additions or changes to Cara's approach?" **]**

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 pressures dissipate, the effective stresses 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 φ'=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.

[In both Teaching Options 1 and 2, Cara's calculations will be discussed in class, accompanied by comments by Ms Moran (both are included in PartA_Calculations_Comments.doc in the supporting material, available only to the instructor, not the students). There can also be a Teaching Option 3, whereby students receive both the narrative and Cara's calculations, which they review in advance before a discussion in class. Students may be given the entire article by Buggy & Curran (2011) or (recommended) only Table 1 from it. The emphasis in Part A is on recognizing modes of failure (learning outcome 1 in Table 2) and selecting appropriate parameter values (learning outcome 3 in Table 2). Part A calculations are straightforward. **]**

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 excavated and even for shallower deposits, such as the 3-m deep alluvium layer in Fig. 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 Fig. 1, only this time she uses 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 3. 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 3. 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), Fig. 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), Fig. 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 (a depth-weighted average was assumed for the cross section to simplify calculations)	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 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.

Step 1. 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:

$$\text{AAOS} = (\sigma_s' - \sigma_f')/\sigma_f' \text{ (expressed as percentage)}, \quad (1)$$

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 linear correlation between C_α'/C_α and log(AAOS) given by the relationship in Fig. 21 by Buggy and Peters (2007) can be used to determine σ_s' and hence h_s; this relationship can be expressed as:

$$C_\alpha'/C_\alpha = 1.85 - 1.09 \times \log(\text{AAOS}). \quad (2)$$

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.

[The aforementioned calculations are included in PartB_Calculations_Comments.doc. Students may be asked to perform some or all the calculations or be given the calculations and asked to perform similar analyses for other cross sections. The calculations in Part B are somewhat involved and require some technical decision making that cannot readily be supported by consulting textbook material. **]**

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.

[As a conclusion of the type "what happened at the end?", the instructor may discuss the actual construction history of the Limerick embankment at Chainage 4+150 (Fig. 22a in Buggy and Curran, 2011) and the monitoring data from a nearby similar cross section at Chainage 4+185 (Figs. 22b-d in Buggy and Curran, 2011). However, such a class discussion may presuppose prior communication between the instructor and the project consultants concerning the significant differences between the actual construction stages and those calculated in Part B (see PartC_Calculations_Comments.doc).**]**

Supporting material

As mentioned, all the calculations corresponding to the tasks described in Parts A, B and C of the case narrative are included in file PartA_B_C_Calculations_Comments.doc, accompanied by ample annotations. The supporting material also includes a PowerPoint presentation with information on the site vicinity, as well as a few selected figures from Buggy and Curran (2011) and two figures from the design report (Alliance, 2006) made available with permission.

It should be clarified that the calculations in the supporting material were prepared by the authors with the information given in the narrative and the project description given by Buggy and Curran (2011), supplemented by clarifications provided by Buggy (2012). Although the authors had access to some design values for the cross section in Fig. 1 (as indicated in Table 3 herein and in the tables of the supporting material), they did not have access to the original analyses performed for the project, nor did they discuss those analyses with the project's consultants. In other words, just as the narrative aims primarily to fulfill instructional goals while remaining faithful to the general design philosophy of the actual project, the calculations are the authors' renditions of the required analyses for the cross section in Fig. 1, sometimes involving simplifications, which are noted. Since the annotated calculations are a 16-page long document, only sample excerpts are included herein, accompanied by some comments on the educational decisions involved in the writing of both the narrative and the supporting material. In order for the excerpts to be distinguished from the interspersed comments, their beginning and end are indicated with bold square brackets. When some text is omitted it is denoted by […]. Figures and tables within brackets are presented with their respective numbers in the supplementary material, e.g. Fig. S1, Table S2, etc.

PART A – Rationale and excerpts from supporting material

Part A is written in a way that gives students some freedom to think what kinds of analyses may be needed for designing a highway embankment founded on soft alluvial soils and what kind of parameters are involved in such analyses. Hence, Ms Moran encourages Cara to first "check out" the problem and think about relevant parameters, before dealing with the specifics of the case study. Students are in a position to do the same, provided that they are given Table 1 from the Buggy and Curran (2011) paper, which is available on the Internet. Part A of the supporting material includes a subset of this table, with Cara's chosen parameters for performing 1-D consolidation settlement analysis. Table 1 from Buggy and Curran (2011) includes some values for C_α, the coefficient of secondary compression. Cara calculates only the primary consolidation settlement, without apparently thinking of secondary compression. Hence, students have the opportunity to comment on this omission (learning outcome 1 in Table 2). Later in Part A, her supervisor explains that minimization of differential settlements is a major requirement for a highway project and calculates herself the secondary compression in the following excerpt from the supplementary material.

Excerpt from the supporting material: Annotated calculation of secondary compression

[Regarding Cara's question concerning the allowable settlements for embankments, Ms Moran stresses that differential settlements must be kept very low. Differential settlements are much more significant close to structures, such as bridges, and, hence, the criteria are stricter there (e.g. < 10mm). Also it is important that settlements do not cause the road gradient to change by too much.

In order to be comprehensive, Cara should also [Note: in addition to calculating the primary consolidation settlement] calculate the secondary compression or creep due to the rearrangement of the soil particles rather than the dissipation of excess pore water pressures. Ms Moran clarifies that it is mainly for calculation purposes that we separate primary consolidation from secondary compression (while in reality they initially overlap) and makes an estimate for Cara's sake.

<u>Computation of secondary compression, ΔH_{sec}</u>

Table 1 on page 19 of Buggy & Curran (2011), B&C (2011) for short, gives the following correlation for the coefficient of secondary compression as a function of water content, w: C_α=0.00018w, which for w=100% gives C_α=0.018. A slightly smaller value is determined for the site-specific correlation given in Fig. 6 of B&C (2011). For these values of C_α, the ratio of C_α/C_c is equal to or less than 0.02, which falls outside the range given in Knappett and Craig (2012) (Table 4.3, C_α/C_c = 0.03–0.08 for clays and silts). According to Mesri and Castro (1987), C_α/C_c falls in a range of 0.02–0.1, while for a majority of inorganic soft clays $C_\alpha/C_c = 0.04 \pm 0.01$ and for highly organic plastic clays $C_\alpha/C_c = 0.05 \pm 0.01$. In order to be on the safe side, secondary compression will be calculated for two values, C_α=0.018 and $C_\alpha = 0.04 \times C_c = 0.04 \times [0.33 \times (1+e_o)] = 0.04 \times 0.74 \rightarrow C_\alpha = 0.03$.

With the values above, secondary compression is calculated as:

$$\Delta H_{sec} = C_\alpha \times H_1 \times \log (t_1/ t_o), \qquad (3)$$

where H_1=thickness of the silt/clay layer after 95% of primary consolidation, t_1= time after start of embankment construction and t_o= time after 95% of primary consolidation [Note: earlier in Part A, Cara has found t_o=4.73yr and H_1=2.14m, but these values are not given here.] At a time of t_1=35 years (the design life of the highway), ΔH_{sec} (C_α= 0.018) = 0.018 × 2.14m × log (35/ 4.73) = 0.033m and ΔH_{sec} (C_α = 0.03) = 0.056m.

According to Buggy & Curran (2011), performance specifications for the project require the projected settlement due to secondary compression to be restricted (page 6). Hence, the calculated secondary compression corresponding to the lower C_α value, which is less than 0.05m, is considered acceptable, while that corresponding to the higher C_α value, which is greater than 0.05m, is not. **]**

This excerpt on secondary compression, a key consideration in the design of the Limerick embankments, is included to illustrate also some decisions that may need to be made in writing a case, when trying to match design calculations to what was actually constructed. Using the site-specific value C_α=0.018, the cross section chosen (where the alluvium is only 3m thick) does not require secondary compression minimization and, hence, does not require surcharge either. However, according to Buggy and Curran (2011), the specific cross section was constructed with a 2.5m surcharge. Hence, the higher C_α values from the literature were used to justify the use of a surcharge. Buggy (2012) later clarified that additional strict criteria for long-term embankment settlement performance may be desired for certain construction methods, such as for semi-rigid pavement construction.

Another opportunity for students to reflect on the chosen parameters (learning outcome 3 in Table 2) is given by Cara's ad hoc choice (from the Internet!) of effective shear strength parameters for the fill material. The only indication Cara could have that the values were plausible is the factor of safety (FoS) calculated for the stability of the embankment slope, i.e. considering potential failure surfaces within the embankment fill. The values used in these initial slope stability calculations are included in Table S2 of the supplementary material, which is reproduced herein as Table 4. The relevant excerpt follows.

Table 4. Values assumed for a first attempt of slope stability calculations, using information from the site investigation (SI) reported by Buggy and Curran (2011) or hypothesized

Parameter [source]	Value chosen
Alluvium	
Unit weight, γ_a [B&C (2011), SI, Fig. 2]	16 kN/m^3
Undrained shear strength [B&C (2011), SI, Table 1 (Limerick Ring Road)]	From c_u/p_o'= 0.3 → at the middle of the 3m layer (p_o'= 19.1kN/m^2), c_u = 6kN/m^2
Fill	
Unit weight, γ_f [Hypothesized as reasonable value for fill]	20 kN/m^3
Shear strength parameters [Ad hoc choice obtained from a textbook assignment]	c' = 25kN/m^2, φ'=20°

Excerpt from the supporting material: Analyzing the stability of the embankment slope using hypothesized shear strength parameters for the fill

[Cara works at home and uses the free student version of Geo-Studio (GEO-SLOPE, 2004) for her calculations. This version allows the user to define only two different materials (which is adequate for uniform fill and alluvium) and circular failure surfaces. […]

For the stability analysis of the embankment slope, the unfactored shear strength parameters from Table S2 [Note: Table 4 herein] give a FoS =2.51. Cara is happy that the FoS is high for the values she has chosen.

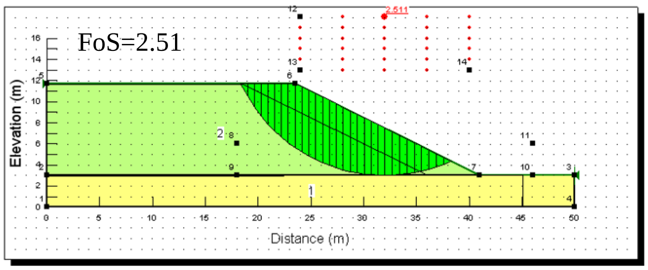

Fig. S1. Results of Geo-Studio analysis for the data in Table S2 (Table 4 herein), considering failure through the fill material only. Geo-Studio input file in supporting material: Part_A_Embankment.gsz]

Cara's choice of shear strength parameters is commented upon by Ms Moran later in Part A as noted below.

[Regarding the shear strength parameters Cara chose for the fill (c′ = 25 kPa and φ′ = 20°), Ms Moran comments that they imply a fine grained fill, probably with a high clay content. Such a material would not be appropriate for use as fill because clay soils are generally difficult to compact as they tend to be in the form of large clumps when excavated and become difficult to work if they become wet. She notes that the fill material generally used in Ireland is glacial till with a wide range of particle sizes and a low plasticity index, I_p, usually less than 20%. The actual fill material used for the Limerick embankments was described as a stoney cohesive material, for which a better choice of shear strength parameters would be c' = 0 and φ' = 35°; this is a more appropriate fill material for the construction of an embankment.]

Part A includes several undrained slope stability analyses for more realistic values for the fill and the alluvium and closes with the following summarizing statements.

[Part A – CONCLUSIONS

- Primary consolidation settlement takes too long to be completed. Vertical drains will be needed to accelerate the consolidation, perhaps requiring a surcharge as well.
- Secondary compression may be an issue.
- Short-term stability for the required embankment height (8.75m) at the cross section considered is marginally adequate as shown by the over-design factor (ODF) (Frank et al., 2004) calculated with soil strength parameters and loads factored by the Eurocode 7 (CEN, 2004) partial factors, as appropriate. An ODF greater than unity indicates that the available margin of safety is greater than that required by Eurocode 7. If the loads are not factored, then for an undrained analysis, the FoS is equal to the partial factor on c_u when the ODF = 1.0. Using the c_u for the fill from Table 3 with a partial factor of 1.4 applied to c_u gives the marginally adequate ODF = 0.99. If a lower partial factor, γ_{cu} = 1.25, were considered acceptable for short-term loading, then a slightly higher embankment could be built for an ODF close to 1. In either case, the extra amount of surcharge that can be applied is either nil (for γ_{cu}=1.4) or minimal (for γ_{cu}=1.25), necessitating construction of the embankment in stages.]

PART B – Rationale and excerpts from supporting material

Part B provides guidance to the students for the kinds of analyses they need to perform, which are broken down into steps and even substeps. This is deemed necessary, because although students can be expected to be familiar with the basic principles underlying the calculations, some of the calculations are more complicated than typical coursework assignments. Hence, Part B focuses on and also goes a step further than learning outcome 2 of Table 2 "Apply methods of analysis already covered in course". The excerpts included herein correspond to the two key calculations for the design of the embankments: the calculation of the surcharge needed for reduction of secondary compression (Step 2) and the combined consolidation – slope stability calculations for the fill heights and hold times of the staged construction (Steps 3a-3c).

Excerpt from the supporting material: Calculate surcharge needed for secondary compression reduction

[As mentioned in the narrative, the amount of surcharge needed for each representative cross section was determined on the basis of the reduction of secondary compression achieved. The approach of creating an overconsolidated soil by surcharging, and hence, in this way, reducing a soil's compressibility from C_c to C_s (the swelling index), is well established. In contrast, reducing C_α by overconsolidation may be a confusing issue, considering (i) that a surcharge is typically used to accelerate primary consolidation, without or with drains, and (ii) statements found in textbooks, such as (Knappett and Craig, 2012, page 137): "It should be realized that the rate of secondary compression cannot be controlled by vertical drains". Alonso et al. (2000) have presented a model that explicitly accounts for the simultaneous contribution of primary consolidation and secondary compression to the total settlement as a function of time (Fig. 15 in Alonso et al., 2000). The same authors remark cautiously on the approach to relating overconsolidation ratio (OCR) to C_α reduction: "such an approach has some limitations from a theoretical point of view, but it provides a good base for achieving results in practice". Perhaps it would be worth modifying the statement from Knappett and Craig (2012) to read: "It should be realized that the rate of secondary compression cannot be controlled by vertical drains *alone, i.e. without surcharge*". It is clear that if only surcharge were used in the Limerick embankment case, i.e. without drains, it would have been impossible to achieve the required secondary compression reduction in the required timeframe of 2 years.

Relationships between reduced values of C_α and OCR may be obtained from the literature or determined specifically for the project soils. The former approach was followed for the Limerick project (Buggy and Peters, 2007: Fig. 21) and later confirmed by tests on the site soils (Conroy et al., 2010). The specific expression used to calculate the reduced C_α was obtained by correlating the ratio of C_α before surcharging to C_α' after surcharge removal with a quantity named the Adjusted Amount of Surcharge, AAOS (Equation 1 in the narrative):

$$\text{AAOS} = (\sigma_s'-\sigma_f')/\sigma'_f \text{ (expressed as percentage)}$$
$$= (\text{OCR (expressed as a ratio)} - 1) \times 100, \qquad (4)$$

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. For the length of the embankment being designed of about 700m that includes cross section 4+150, the calculated AAOS values ranged from 20% to 40% [Alliance (2006): page 9], corresponding for the majority of cross sections to a surcharge of 2.5m.

For a surcharge of 2.5m, the maximum embankment height at cross section 4+150 is 11.25m, hence the vertical effective stresses at the middle of the alluvium layer are:
σ_s'= 17 x 1.5 – 0.5 x 9.81 + 21kN/m^3 × 11.25m
= 20.6kN/m^2 + 21kN/m^3 × 11.25m = 256.9kN/m^2
After surcharge removal: σ_f'= σ_1'= 20.6kN/m^2 + 21kN/m^3 × 8.75m = 204.4kN/m^2
Then, AAOS = $(\sigma_s'-\sigma_f')/\sigma'_f$ = (256.9-204.4)/204.4 = 26%

For a surcharge of 2.75m, the maximum embankment height is 11.5m, hence:
σ_s'= 20.6kN/m^2 + 21kN/m^3 × 11.5m = 262.1kN/m^2
and AAOS = $(\sigma_s'-\sigma_f')/\sigma'_f$ = (262.1-204.4)/204.4 = 28%

From Fig. 21 of Buggy and Peters (2007) (and, easier, using Equation 2 in the narrative), the AAOS values of 26% and 28% correspond to C_α'/C_α ratios of 0.31 and 0.27, respectively. By selecting a surcharge of 2.75m and C_α' = 0.27C_α, and assuming the conservative value for C_α = 0.03 [based on the C_α/C_c ratio of Mesri and Castro (1987)], the reduced value for secondary compression is calculated as:

$$\Delta H_{sec} = C_\alpha' \times H_1 \times \log (t_1/ t_o)$$

where H_1= thickness of the silt/clay layer after 95% of primary consolidation (H_1=2.06m) [Note: this value is calculated with the site-specific C_c and not with the value assumed by Cara in Part A], t_1= time after start of embankment construction, t_o= time after 95% of primary consolidation. For t_o = 19 months = 1.58 years [Note: when using drains, 95% of primary consolidation is completed in 19 months] and after t_1 = 35 years, the secondary compression is:

<u>ΔH_{sec}</u> = (0.27× 0.03) × 2.06m × log (35/ 1.58) = <u>0.022m</u>, which is acceptable. **]**

The next excerpt includes parts of Steps 3a-3c, i.e. the combined consolidation – slope stability calculations for the fill heights and hold times of the staged construction. It is relevant to note that the topic of stability evaluation during staged construction was the subject of the 22nd Terzaghi Lecture delivered in 1986 (Ladd, 1991). The slope stability calculations are executed using the paid version of Geo-Studio, because it permits definition of planar failure surfaces, which were found to give lower factors of safety than circular failure surfaces. Since the paid version also allows several layers with different material properties to be defined, some analyses were performed with the design values for sublayers within the alluvium (Alliance, 2006) in order to investigate the effect of the higher c_u of the slightly overconsolidated soil close to the ground surface. Specifically, c_u varied as follows (0m is ground surface): 0-0.8m: 35kN/m^2, 0.8-1.5m: 23kN/m^2, 1.5-2.4m: 15kN/m^2, 2.4-3m: 30kN/m^2: these values give the depth-weighted average of 25kN/m^2 in Table 3. It should be noted that the lower part of the embankment was a 0.5m-thick gravel drainage layer, which was ignored in the slope stability calculations.

Excerpt from the supporting material: Undrained slope stability analyses for various embankment heights
[Step 3a
In Part A, it was determined that an embankment height of 8.75m can be constructed with a marginally adequate margin of safety when c_u is somewhat less (20kN/m^2) than the weighted average of 25kN/m^2. Therefore, it is decided to start with a maximum height of h_1 = 8m for Stage 1.

<u>Loading Stage 1</u> Using the undrained shear strength parameters in the fill and in the different depths in the alluvial soil, divided by a partial factor of 1.4, and with a failure mechanism involving planar failure surfaces gives ODF=1.10.

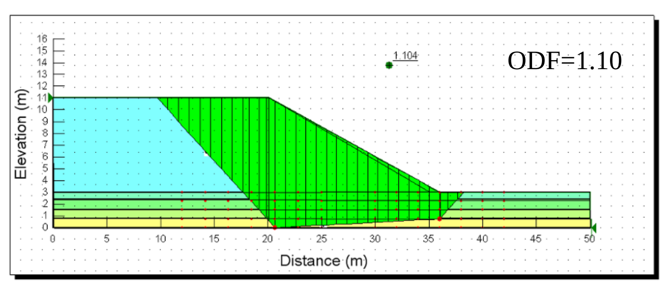

Fig. S8: Slope stability analysis for 1st loading stage. Geo-Studio input file: Part_B_StageI_4LayerAlluvium.gsz

The calculations for circular failure surface give a higher ODF=1.24 (Geo-Studio input file in supporting material: Part_B_StageI_4LayerAlluviumCircle.gsz). To simplify the calculations, from now on, slope stability analyses will be performed for the depth-averaged uniform shear strength value for the alluvium c_u= 25 kN/m^2, and planar failure surfaces.

Loading Stage 1 The analysis is repeated for a uniform alluvium layer, with c_u= 25 kN/m^2/1.4 = 17.8 kN/m^2, giving ODF=1.03 (which is not too different from, and lower than, the value calculated considering the variation in the c_u of the alluvium, i.e. ODF=1.10).

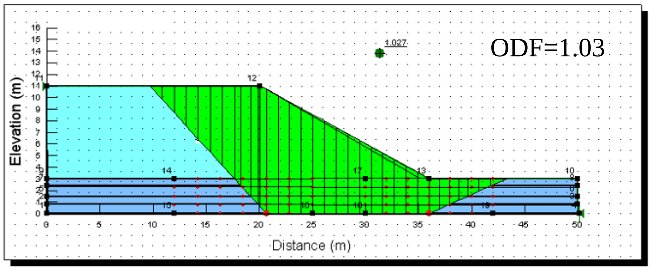

Fig. S10. An 8-meter high embankment over an alluvium layer of 3m at its initial undrained shear strength. Geo-Studio input file: Part_B_StageI_UniformAlluvium.gsz

Step 3b

For an embankment of height 8m, the max $\Delta\sigma'$ (at U_r=100%) due to the fill is equal to 168kN/m^2. The increase in vertical effective stress at the middle of the 3m alluvium layer (which has an initial vertical effective stress p_{oi}'= 20.6kN/m^2) is assumed to be proportional to the degree of consolidation due to radial drainage only. For this assumption, the vertical effective stress, p_o', at the middle of the layer as consolidation proceeds is:

$$p_o' = p_{oi}' + [U_r(t_{h1})/100] \times \max \Delta\sigma', \qquad (5)$$

where $U_r(t_{h1})$ is the degree of consolidation considering only radial drainage at Stage 1 hold time t_{h1}. […] Equation (5) gives the results shown in Table S5 for the increase in the undrained shear strength, c_u, with time. Equation (5) can be improved upon by considering the combined degree of consolidation, including both radial and vertical drainage.

Table S5. Undrained shear strength values for the alluvium for various hold times of an 8-meter high fill

Stage 1 hold time t_{h1} (months)	Time factor T_r	Degree of consolidation $U_r(t_{h1})$	Vertical effective stress p_o' (kN/m^2)	Undrained shear strength c_u(kN/m^2) =0.3 p_o'
2	0.09	0.27	65.6	20
4	0.18	0.47	100	30
6	0.27	0.61	123.1	37
9.4	**0.42**	**0.77**	**150**	**45**

Step 3c

The shear strength increase after 6 months provides a margin of safety that does not satisfy Eurocode 7 since ODF=0.9 for c_u=37 kN/m^2/1.4=26.4 kN/m^2

The search for the adequate c_u yields: c_u=45 kN/m^2/1.4=32.1 kN/m^2 giving a satisfactory ODF = 1.07.

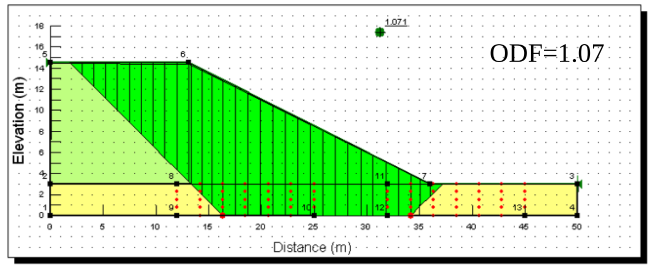

Fig. S12. An 11.5-meter high embankment over an alluvium layer of 3m at the undrained shear strength it has acquired after being loaded by a 8-meter high fill for 9.4 months. Geo-Studio input file in supporting material: Part_B_StageII_UniformAlluvium.gsz

The critical circular failure surface gives ODF = 1.18, i.e. again a higher value compared to the critical planar failure surface.

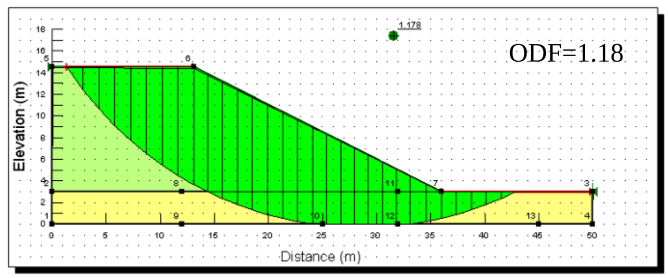

Fig. S13 Same material parameters as in Fig. S12, different definition of failure surface. Geo-Studio input file: Part_B_StageII_UniformAlluviumCircle.gsz]

Part B closes with the summarizing statements below, followed by some comments on Part C.

[Part B – CONCLUSIONS

- The required surcharge height was calculated on the basis of some hypothesized low desired value for secondary compression. This surcharge was equal to 2.75 m, on top of an 8.75m embankment.
- Based on the initial undrained shear strength of the alluvium (determined on the basis of CPT results), a height of 8m is safe for Stage 1 construction. This result remains to be confirmed by monitoring measurements during construction.
- Based on undrained slope stability analyses and considering the improvement in the undrained shear

strength (obtained using a correlation with the vertical effective stress), after about 9.5 months of hold time for Stage 1, the remaining 3.5 m of fill can be added in Stage 2. This result remains to be confirmed by monitoring measurements during construction. In addition, these calculations need to be refined to take into account the filling rate of Stage 1, projected to be around 1m per week.

PART C – COMMENTS

The conclusions reached in Part B (Stage 1: 8m & 9.4 months, Stage 2: 11.5m) do not agree with how the embankment was actually constructed. A similar disagreement is found between how the embankment was intended to be constructed according to the design report (Alliance, 2006) with Stage 1: 10m & 6.4 months, Stage 2: about 11.5m & 24 months, then remove surcharge, and how it was actually constructed [(Buggy and Curran, 2010), Fig. 22(a)] with Stage 1: 4m & 5.5 months, Stage 2: 10m & 7.5 months, Stage 3: 11.5m & 7 months, then surcharge is removed. The discrepancy between the Stage 1 heights calculated herein and in the design report are due mainly to the higher margin of safety adopted in the calculations herein through using the Eurocode 7 partial factor of 1.4 for the undrained shear strength compared to the overall FoS=1.25 for undrained analyses used in the design report (which was completed before the implementation of Eurocode 7). The discrepancy between the as-designed and as-constructed heights were due to (a) earthworks logistics and materials supply and (b) adjustments necessitated by the monitoring results (Buggy, 2012), in the context of the observational method which was adopted in this project. For example, Buggy and Curran (2011) report that the ratio of the lateral toe movement to embankment crest settlement, $\Delta Y/\Delta S$, for cross section 4+150 rose rapidly to the local maximum value of 0.4 during Stage 1 filling. **]**

CONCLUDING REMARKS

When instructors wish to use a case study for general purposes, they have many choices. However, when they intend to use cases to achieve specific higher level learning outcomes (i.e. past the “recall” level), the case study must be written with this specific goal in mind, in order to allow active involvement of the students with the case material. The case study presented herein was written as an example of the latter kind. To this end, it consists of a 5-page long case narrative, which is written for the students and which guides them to decide on the relevant methods of analyses and the required soil parameters. The narrative is supplemented by a 16-page long supporting document, which is written for the instructor and includes annotated calculations and comments.

The case study developed is based on a project involving embankments constructed on soft fine grained material. The case narrative centers around the two pivotal geotechnical issues for the project: (I) excessive settlements, which require (Ia) vertical drains to speed up consolidation and (Ib) a surcharge to reduce secondary compression, and (II) low undrained strength, which necessitates staged construction of the full height of the embankments plus the required surcharge. Such problems can be solved by students using foundational concepts and basic theories of geotechnical engineering. Hence, it is expected that the case will be appropriate for most introductory geotechnical engineering courses. The supporting material is expected to save instructors’ time and facilitate use of this case in a geotechnical course.

As an encouragement to colleagues contemplating the time commitment required for the *development* of a case study for instruction, the authors would like to share some unexpected benefits they received. For the second author, whose expertise is environmental geotechnics, the development of the case provided a sample of vicarious consulting experience in classical geotechnical topics. For both authors, it offered an opportunity to rethink the mainstays of geotechnical courses, such as consolidation settlement and secondary compression, and how they are applied in practice. Both authors look forward to proposed additions to (and disagreements with!) the supporting material from instructors contemplating using the Limerick case in geotechnical courses.

ACKNOWLEDGEMENTS

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