Settlement calculations of a motorway embankment and comparison with field measurements: An education case study

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ABSTRACT: This paper focuses on the presentation of a case study that is suitable for instruction in the geotechnical engineering field, mainly because settlement calculations for a motorway embankment are accompanied by field measurements during a two-year period that includes both the construction period and the operation of the motorway. The examined case study refers to a reinforced embankment of 12.3 m height founded on highly compressible alluvial deposits. Settlement calculations are performed using conventional methods, using the theory of one-dimensional consolidation. The settlement-time curves resulting from the relevant geotechnical calculations performed for each one of the intermediate construction stages are compared with the corresponding field measurements. In all the examined cross sections, the calculated settlements are higher than the measured settlements; overprediction ranges from 36% to 207%. The case study provides an opportunity to compare methods taught and used in practice and to discuss variability expected in practice.

Keywords: Monitoring, Settlement, Consolidation

1 Introduction

The selection of the case study presented herein is consistent with research-based principles for learning from the education literature. Specifically, the principle that concerns students' motivation, which according to Ambrose et al. (2010) "determines, directs, and sustains what they do to learn". Ambrose et al. (2010) explain that motivation is related to students' personal investment and the subjective value of the learning goal and the task at hand. In order to influence the subjective value of the learning goal, they recommend that instructors make educational material more interesting to students by selecting authentic, real-world tasks to which ideally they can relate. To this end, the case study selected and presented herein concerns a significant settlement of a motorway embankment, which students can approximate with knowledge from an introductory soil mechanics course. What is more, its location is close to a well-known locale from Ancient Greek History, Thermopylae, also known in English as "Hot Gates".

The notable features of the selected project are summarized as follows. 1) Fixed elevations and space constraints resulting from the existing motorways to be connected, which necessitate some steep-slope (3:2) (height:base) embankments that will have to be reinforced with geosynthetics. 2) The soils are recent quaternary alluvial deposits of considerable thickness, predominantly fine-grained over at least the top 30-35 meters, which are expected to necessitate construction of the embankments in stages. 3) The significant heterogeneity of soils is a source of uncertainty for the soil profiles at cross sections further away from the boreholes. Due to (2) and (3), monitoring is a key element for the construction of the embankments. The combined historical-geological interest of the project location, which is tightly linked to the significant magnitude of consolidation settlements, and the 2-year long monitoring record make this project a good candidate for an education case study.

In previously published educational case studies, we chose an 8.75 m high road embankment with a calculated settlement of 0.99 m (Orr & Pantazidou, 2013) and an 8 m high railway embankment with a calculated settlement 0.64 m (Xenaki et al., 2016). The 8.75 m embankment necessitated construction in stages, whereas the 8 m embankment was to be constructed in a single stage. In the present paper, we focus on a 12.3 m high motorway interchange embankment with a calculated settlement of 1.34 m, which again necessitated staged construction. Motivating questions for the use of this case study in instruction include “why is the settlement so large?” and “do we feel comfortable or nervous with this large calculated settlement for the specific locale?”. The main goal of this paper is to highlight the relevance of theory taught in courses to practice and relate depositional history to soil parameters. A second goal is to attempt to draw lessons from comparisons of calculated and observed settlements at several cross sections of the interchange.

2 Geology and design soil layers

2.1 Project area

Geological observations and the pertinent data from the geotechnical investigations (execution of sampling boreholes) indicate that the interchange passes through recent quaternary deposits which have a significant thickness. Results from a total of 52 boreholes were evaluated, 18 of which were drilled for an earlier alignment of the motorway and are not as close to the final location of the embankment. The maximum borehole depth is 45.35 m, while most boreholes reach a depth of 30-35m.

More specifically, alluvial deposits (al) are encountered in the project area, characterized by significant heterogeneity in both horizontal and vertical directions. The alluvial deposits in the study area consist either of clay and clayey silts, or of silty sands, or of sand and gravel.

Based on a) the pre-existing geological report, b) the macroscopic description, c) the grain size distribution curves, and d) the results of laboratory tests on characteristic soil samples from the relevant boreholes, the geological formation of the alluvial deposits at the project area is divided into five distinct geotechnical formations (I to V) (see Figure 1). Specifically, in the upper 20 m to 25 m, very soft to soft clayey-silty formations I and II (CL, CH, ML, CL-ML) are encountered, characterized by low values of the Standard Penetration Test (SPT) blow counts (N_{SPT}<12) and therefore by poor mechanical properties, with interlayers of sand and locally gravel (formation III). At greater depths, clayey-silty formations (CL, ML, CL-ML) (formations IV and V) are found with higher SPT blow count values (N_{SPT}>12) and better mechanical properties, which improve even further for depths greater than 35.0 m. At the project area the groundwater level is near the surface, therefore the soil is saturated.

2.2 Broader geological background

Zooming out from the project area, we see that the interchange is located in the Sperchios delta plain, about 10 km away from the current shoreline of the Malian Gulf. The Sperchios basin is well known in history from the battle that took place in 480 BC between ancient Greeks and Persians at Thermopylae (also known in English as “Hot Gates” from the hot springs located by the mountain’s foothills). At the time of the battle, the Thermopylae Pass is described by Herodotus as a narrow strip of land between Mount Kallidromon and the Malian Gulf Sea. The identified area of the battle is now more than 5 km away from the sea (Vouvalides et al., 2010). The alluvial sediments carried by Sperchios River have been filling the basin through the centuries – and in an asymmetric fashion, i.e. a significant part of the Sperchios delta plain was established earlier compared to the part near Thermopylae (Pechlivanidou et al., 2014) – and now the site of the battle location is the boundary of the mountain foothills and the alluvial plain. This high sediment discharge is due to the erodible sedimentary rocks (flysch) rich in clay minerals to the south of Sperchios River (Pantazidou et al., 2021; Marinos, 2025). In other words, history together with geology give us clues to expect young compressible sediments in the project area.

3 Design components and geotechnical parameters of soils

The examined interchange has an approximate total length of 1.5 km. The maximum height of the embankments is equal to 14.2 m. Based on the geometric characteristics of the embankments and on the geotechnical conditions, the embankments are divided into three categories depending on the implemented construction methodology: a) embankments with slope inclination 2:3 (height:base), b) embankments with slope inclination 2:3 (height:base) with suitable basal reinforcement using geogrids and c) reinforced embankments with slope inclination 3:2 (height:base) using geogrids in combination with gabions at the slope faces.

The main geotechnical calculations performed for the design of the embankments include the following analyses at the most critical cross sections (Salgado, 2007; Barnes, 2005):

a) Slope stability analyses and more specifically:

- The initial slope stability analyses were focused on the soil bearing capacity, due to the presence of alluvial silty-clayey layers characterized by low shear strength parameters, which, in combination with the significant height of the embankments, makes it impossible to construct them in one stage. Therefore, staged construction of the embankments was proposed in the pertinent geotechnical study. The height of each intermediate construction stage was selected in such a way so that the soil bearing capacity, depending mainly on the undrained shear strength of the foundation layers, is adequate. For the first construction stage, slope stability analyses were performed using the initial undrained shear strength values of the soil layers, whereas for the subsequent construction stages increased undrained shear strength values were taken into consideration to account for the soil improvement due to preloading. The time-dependent improvement of undrained shear strength, Δc_u, of the foundation soil layers is affected by the imposed preloading (height of embankment) and the degree of consolidation which has been achieved during the preloading period. Prefabricated vertical drains are constructed to accelerate the consolidation.

  The height of each construction stage has been selected by taking into consideration the most critical section (cross section 195, see later Table 3), where the silty-clayey layer characterized by high compressibilty and low undrained shear strength values has the maximum thickness.

  The maximum shear strength improvement is achieved under the central part of the embankment, whereas under the slopes of the embankment no shear strength improvement was accounted for in the stability analyses.

  The above mentioned stability analyses for the intermediate construction stages refer to short-term conditions and were performed only under static loading conditions.

- At the final construction stage, at which the total height of the embankment constituting the permanent earthwork was completed, slope stability analyses were performed under both static and seismic loading conditions. It is noted that at the final stage, the internal slope stability is also critical due to the significant embankment height. For the reinforced embankments with steep slopes (3:2, height:base), apart from rotational analyses, translational analyses in predefined surfaces along the geogrid layers (two-part wedge mechanism) were also performed. The above analyses also validated the adequacy of the selected geogrids in terms of nominal tensile strength.

The traffic load in the slope stability calculations of the final costruction stage is modelled by applying a distributed load on the crest of the embankment equal to p=20 kPa and p=10 kPa, for the case of static and seismic loading conditions, respectively.

The slope stability analyses were performed with the limit equilibrium method, using Larix-4S software (v. 2.21-Cubus) for the embankments with 2:3 (height:base) slope inclination and ReSSA software (V. 3.0, Adama Engineering, Inc. Newark, USA) for the embankments with 3:2 (height:base) slope inclination.

b) Calculation of the soil settlements due to the construction of the embankment.

Due to the nature of the encountered alluvial formations at the project area, where silty-clayey layers are predominant, in combination with the high groundwater level, the development of both immediate and consolidation settlements are expected to occur under the imposed embankment load. The calculated immediate settlements are expected to be completed during the construction of the embankment, whereas the consolidation settlements are time dependent.

c) Evaluation of liquefaction susceptibility of the bearing soil layers.

The relatively high groundwater level, in combination with the existence of loose sandy and silty soil layers at the area of the embankments, creates the conditions for possible initiation of liquefaction phenomena under seismic loading conditions. The relevant assessment methods indicate only the occasional existence of liquefiable layers of small thickness. Therefore this mode of failure is not taken into consideration for further analysis in the presentation of the current case study.

Groundwater level is located at 0.5 m to 1.0 m depth below the ground surface at the area of the embankment.

For the construction of the main part of the embankment, it is suggested that mainly coarse-grained crushed materials are used. The embankment material contains also fine material (d<0.063 mm) up to 35%. The slope stability analyses and the settlement calculations of the embankment are carried out assuming the following geotechnical parameters for the material of the embankment: unit weight $\gamma=20$ kN/m^3, angle of internal friction $\varphi=32^o$, cohesion c=5 kPa and elasticity modulus $E_s=50$ MPa. The assumed shear strength values are validated by the Contractor by performing laboratory tests on compacted samples of the embankment material.

The determination of the detailed soil stratigraphy along the embankment as well as the determination of the geotechnical design parameters of the encountered soil formations have resulted from the geotechnical evaluation of the investigation data, based on the results of in situ tests (Standard Penetration, SPT) and laboratory tests. Table S1 in the OnlineSupplement includes the detailed list of the delineated sublayers with the respective design parameters. The heterogeneity of the encountered alluvial deposits is strongly depicted in the embankment longitudinal section given in the Supplement (Figure S13). The soil stratigraphy at the critical cross sections has been derived from the above mentioned longitudinal section.

In the context of this education case study, only settlement calculations are presented in detail and compared with monitoring results.

4 Calculated and measured embankment settlements

In the pertinent geotechnical study the following assumptions were made for settlement calculations:

- Regarding consolidation settlements, it is assumed that the deformations of the compressible layers will occur only in one dimension. Therefore, the theory of one-dimensional consolidation is used.

  The laboratory oedometer test results for the silty-clayey layers undergoing consolidation indicate low values of preconsolidation stress, p'_c, compared to the effective overburden stress. However, low SPT values ($N_{SPT} < 4$) were frequently encountered near the surface. Thus it is considered that the surface clayey layer Ia is normally consolidated (overconsolidation ratio OCR=1.0). To account for the improvement of geotechnical parameters with depth, slightly increased OCR values for the underlying layers (layers II, IVa and IVb) varying from 1.1 to 1.5 were assumed.
- Apart from the total magnitude of consolidation settlement (ultimate consolidation settlement at time $t=\infty$), the rate of consolidation settlements is also of great significance in the geotechnical design of the embankment. In order to determine whether the operation of the motorway will be affected by the magnitude of the remaining consolidation settlements, the Owner of the Project has set an available time period equal to 9-12 months for the completion of the embankment construction. In case the remaining settlements after the above time period are greater than 15.0 cm (maximum value specified by the Owner of the Project), the installation of vertical wick drains is required in order to increase the rate of consolidation settlements.
- Settlement calculations at the critical cross sections were carried out by taking into consideration a base improvement layer with thickness varying from 0.5 m to 1.0 m, depending on the thickness of unsuitable material that has to be removed for the foundation of the embankment.
- Apart from the embankment load, a traffic load equal to 20 kPa is also taken into consideration for the settlement calculations, as already mentioned in the description of stability analysis.
- The influence depth for the calculation of settlements is considered as the depth at which the imposed foundation load (reduced with depth) is equal to 20% of the effective overburden stress.

- The encountered alluvial deposits (al) are characterized by significant heterogeneity and the existence of sandy interlayers inside the silty-clayey layers is frequent. These sandy layers were neglected when calculating the consolidation settlements. Therefore, a conservatively high thickness of layers undergoing consolidation settlement is considered in the pertinent calculations, the priority being to avoid erring on the side of underprediction.
- Due to the above mentioned heterogeneity, the soil stratigraphy may differ not only along the embankment, as depicted in the longitudinal section of the Supplement (Figure S13), but also across each examined embankment section.
- During the construction of the embankments, settlements were regularly measured (on a weekly basis) making it possible to validate the assumptions involved in the calculations. More specifically, settlement calculations were performed for each one of the intermediate construction stages and were evaluated by comparing them with the measured values.
- In the case of the examined cross sections, possible discrepancies between measured and calculated settlements do not affect significantly the dimensioning of the project, with the exception of the design for the required prefabricated wick drains. The selected grid of the drains is dependent on both the assumed degree of consolidation, related to the coefficient of consolidation, C_v, and the magnitude of consolidation settlement. However, faster consolidation also means faster increase of undrained shear strength allowing an earlier placement of the next layer and an earlier completion of the project.

Figure 1 shows the layers encountered at cross section 195 (at chainage 2+850) that will be discussed in detail for the purposes of this education case study, and the design parameters necessary for the calculation of settlements. Two more cross sections are included in the Supplement (Figures S2 and S3). The closest borehole to cross section 195 is ED-S1-BE06 (see Figure 13 in OnlineSupplement), drilled to a depth of 30 m. The depicted cross section reaches a depth of 52.5 m, which corresponds to the influence depth of the load (to a level of 20%, as already mentioned). Clearly, there is scanty information below a level of 30 m.

4.1 Simplified consolidation settlements

For the purposes of this education case study, the OnlineSupplement includes a case narrative where an undergraduate student performs simplified calculations of consolidation settlements using the data shown in Figure 1. Two sets of detailed calculations are included in the Supplement. For a load constant with depth, consolidation settlement is equal to 1.39 m. For a load attenuating with depth at a 2:1 (height:base) slope, consolidation settlement is equal to 1.08 m.

4.2 Settlement calculations from the project report

Settlement calculations due to the construction of the embankment were performed with the theory of one-dimensional consolidation, implemented in a Microsoft excel calculation sheet. In order to obtain more precise results regarding both immediate and consolidation settlements, each encountered layer is divided into sub-layers of smaller thickness. Settlements are calculated below the embankment axis and below the toe of the embankment. The stress distribution below the embankment is according to Osterberg (1957).

Apart from the magnitude of the settlement, the rate of consolidation is also estimated. As already mentioned, in order to satisfy the criteria regarding the allowable remaining settlement determined by the Owner of the Project (less than 15 cm) after the construction period (from 9 to 12 months), the installation of wick drains is required. Therefore the rate of consolidation is calculated by taking into consideration both vertical and radial drainage conditions. Although the time progression of the settlements is of great importance for staged construction, this paper focuses mainly on the total magnitude of consolidation settlement, to match the focus of the case narrative.

Table 1 includes the pertinent geometric parameters and the calculated settlements of the embankement axis for all the construction stages for cross section 195 in Figure 1. In the intermediate construction stages no surcharge is applied, whereas in the final stage a surcharge of 1.0 m is applied. Total consolidation settlement is equal to 1.18 m. For additional insight into the development of consolidation settlements (S_{cons}), the contribution of each layer is given separately ($S_{cons,i}$) and is also quantified as a percentage of the total consolidation settlement ($S_{cons,\,i}/\ S_{cons,\,tot}$ (%)). These results show that the contribution of the surface layer in terms of consolidation settlement is predominant. It is also clear that immediate settlements are only a small part of the total settlements for all construction stages.

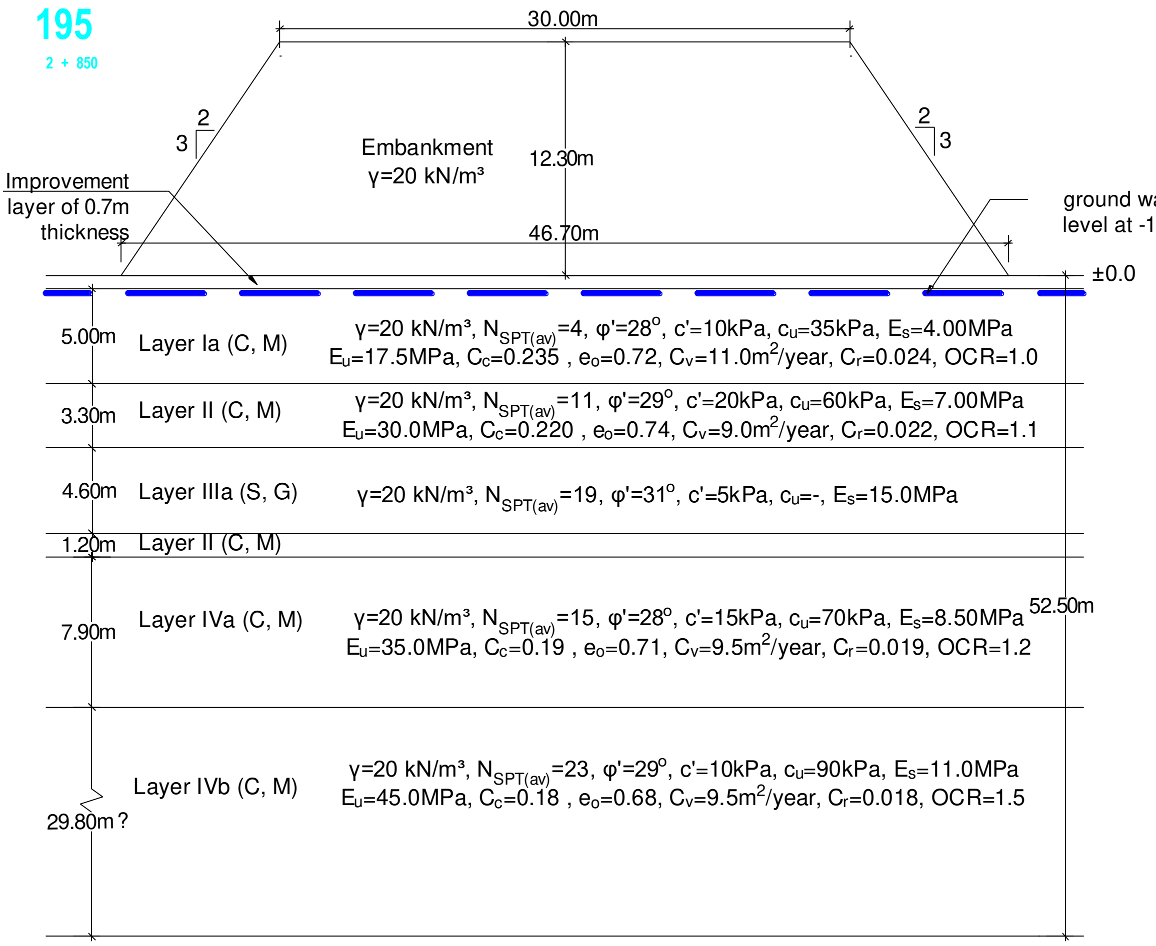

Figure 1. Characteristic cross section for settlement calculations

4.3 Monitoring results and comparison with settlement calculations

This section presents the measurements from three settlement gauges at cross section 195, ST10.2, ST11.2, and ST12.2, placed at the left edge, at the axis and at the right edge of the embankment, respectively. Details of all the monitoring instruments used and the dates of the staged construction are given in the OnlineSupplement. The waiting period between two stages was either 2 months or until stabilization of the measurements from all monitoring instruments.

Table 2 includes the calculated settlements at the embankment axis and the measurements from the three devices. Calculated settlements are consistently bigger than measurements. The overprediction trend decreases at each subsequent stage from 276% to 42%.

The comparison of measured and calculated settlement values as a function of time is shown in Figure 2. More specifically, Figure 2 shows a) embankment height vs. time from the start of construction b) settlement data provided by the Contractor vs. time from the start of construction for settlement gauges ST10.2, ST11.2, and ST12.2 and c) estimated settlement-time curves derived from the pertinent geotechnical calculations for the four construction stages in Table 2. (i.e. for embankment heights of 4.0 m, 8.0 m, 11.5 m, and for the final height of 12.3 m or 13.3 m with the surcharge).

It should be noted that in the settlement diagram of Figure 2, the depicted blue curve is theoretical and has been derived by combining the individual calculated settlement vs. time curves, each one corresponding to the intermediate embankment heights. The settlement calculations, in terms of time, were made taking into account the actual length of the installed prefabricated wick drains (based on the data provided by the Contractor), which for the examined section is the same with the proposed length of the geotechnical study and equal to 20 m. The "beginning" of time in the theoretical blue curve is indicative and was selected to correspond to the time of imposition of the full embankment load of the first construction stage. The superposition of the individual curves (dashed blue lines) is also indicative to show the transition to the successive theoretical calculation curves of each intermediate stage.

Table 1. Calculated settlement values at the cross section 195

Construction stage	Embankment height	Embankment geometry		Influence depth	Calculated settlements at the embankment axis (cm)					
		Base width, A_1	Crest width, A_0		$S_{immed.}$	Consolidation settlement per layer, S_{cons}				Total
	(m)	(m)	(m)	(m)		Layer	$S_{cons,\,i}$	$S_{cons,\,i}/S_{cons,\,tot}$ (%)	$S_{cons,\,tot}$	
1st	4.0	46.7	41.3	28.4	3.2	Ia	34.9	65.1	53.6	56.8
						II	10.8	20.1		
						IIIa	0.0	0.0		
						II	2.1	3.8		
						IVa	5.3	9.9		
						IVb	0.6	1.0		
2nd	8.0	46.7	35.8	42.8	9.6	Ia	50.3	57.3	87.8	97.4
						II	18.0	20.5		
						IIIa	0.0	0.0		
						II	3.9	4.4		
						IVa	13.8	15.7		
						IVb	1.9	2.1		
3rd	11.5	46.7	31.0	50.8	13.9	Ia	59.2	53.5	110.7	124.6
						II	22.5	20.4		
						IIIa	0.0	0.0		
						II	5.1	4.6		
						IVa	19.6	17.7		
						IVb	4.3	3.8		
Final	12.3 plus surcharge of 1.0m	46.7	30.0	52.5	15.9	Ia	60.8	51.5	118.0	133.9
						II	23.9	20.3		
						IIIa	0.0	0.0		
						II	5.5	4.7		
						IVa	21.9	18.6		
						IVb	5.9	5.0		

Table 2. Comparison of calculated and measured settlement values at cross section 195 at the end of each construction stage

Construction stage	Embankment height	Duration (from start of construction)	Calculated settlements at the embankment axis (cm)			Measured settlements (cm)		
	(m)	(days)	Immediate	Consolidation	Total	ST10.2 left	ST11.2 axis	ST12.2 right
1st	4.0	88	3.2	53.6	56.8	15.8	15.1	16.0
2nd	8.0	161	9.6	87.8	97.4	42.2	46.6	43.4
3rd	11.5	245	13.9	110.7	124.6	69.9	73.0	72.7
Final	12.3 plus surcharge of 1.0m	406	15.9	118.0	133.9	92.3	94.4	90.7

Figure 2 shows that toward the end of each construction stage, measured settlements have almost stabilized. Stabilization of settlements and measured settlements not exceeding calculated values were the two criteria to proceed to the next construction stage. The evolution of settlement with time shows again that the discrepancy between measured and calculated values is more pronounced at the early construction stages and becomes less significant when reaching the final stage of construction.

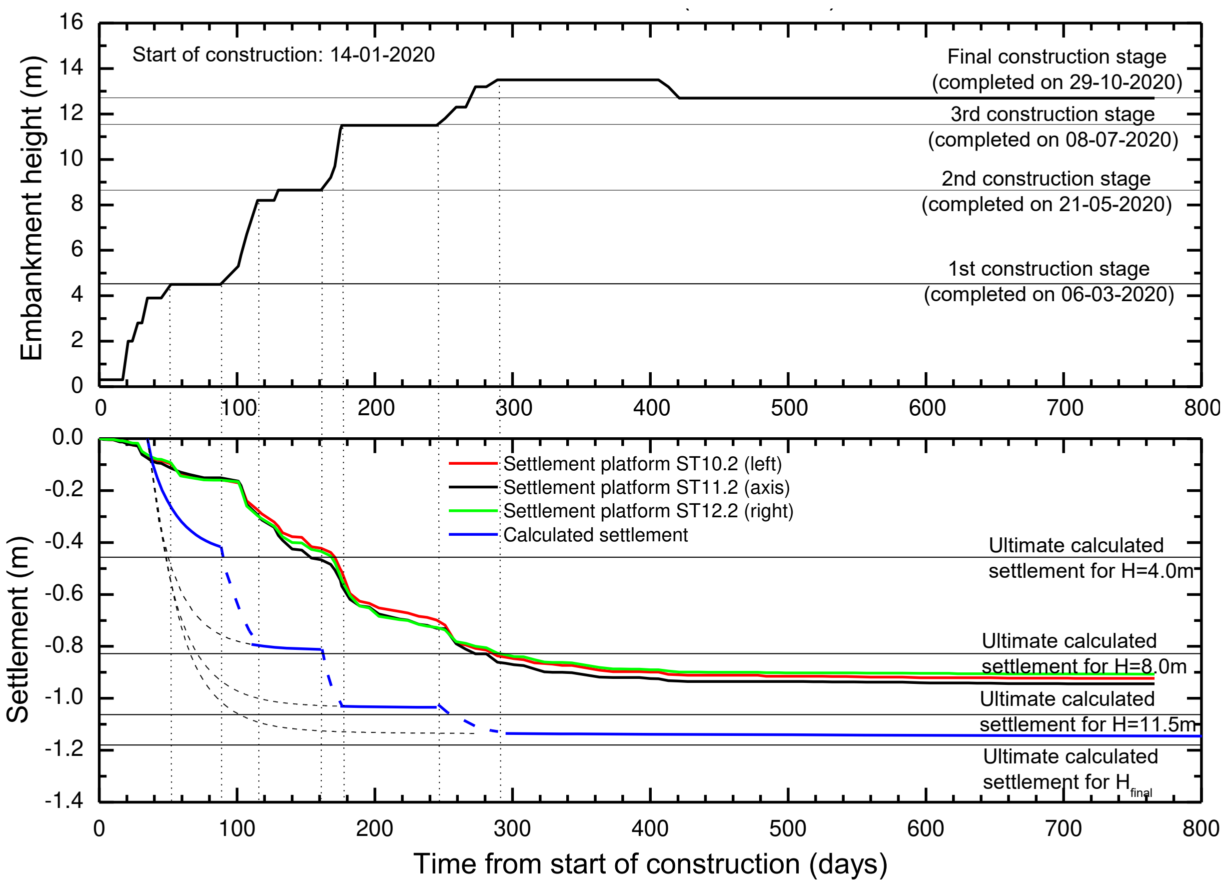

Figure 2. Progression of embankment height, calculated and measured settlements at cross section 195

Table 3 shows measured vs. calculated settlements at all nine sections of the main branch of the motorway embankment. Calculated settlements again are higher. The last column of Table 3 includes the overprediction, computed as (calculated value – measured value)/(measured value) and found to vary from 36% to 207%. The highest overpredictions occur at the two shortest embankments (cross section 79, height: 4.10 m; cross section 301, height: 6.30 m).

4.4 Discussion

Differences between measured and calculated settlements raise two issues for discussion in the classroom (and beyond). The first issue is where should we look for reasons for the differences and what–hypothetically or realistically– could we have done differently. Reasons may concern calculation methods, soil profile and soil parameters. An improvement of the calculation methods is to consider two-dimensional consolidation, but this is not expected to result in large reduction of settlement. Another improvement concerns considering a stress distribution produced by methods better suited for the project soils, i.e. not from elasticity assuming a uniform profile. Again, this is not expected to make a big difference. In fact, for weaker surficial soils, under the embankment axis it would produce larger increases of effective stress (but smaller increases under the toe) and, hence, even larger settlements. A third possibility that combines geometry and stress distributions may be more germane to the specific case study, which deviates from plane strain, is to account for the three-dimensional effects related to how the embankment was actually constructed. In reality, the embankment length is limited, especially at the early stages of the construction, depending on the requirements of the construction site. The embankment has a total length of 1.5 km, but it is interrupted by two bridges, where the interchange goes over the motorway, between cross sections 157(T1) and 185, and over the railroad, between cross sections 239(T3) and 264(T4) (see OnlineSupplement, e.g. Figures S6, S12, S13, S14). Cross section 195, almost 50 m wide, is at a distance of 80 m from the 90-m long bridge going over the motorway and 240 m from the 75-m long bridge going over the railroad. This is another reason why cross section 195 was selected for this case study (in addition to the significant thickness of the underlying silty-clayey layers already mentioned), instead of cross section 239(T3), where the embankment is highest (14.2 m), which is only 20 m away from the bridge going over the railroad.

Table 3. Calculated and measured settlements at nine cross sections of the 1.5-km long embankment

Measurement device code	Cross section	Chainage	Position of measurement device	Embankment height (m)	Settlement values (cm)		Over-prediction
					Measured	Calculated	
M2270	79	2+270	axis	4.10	21.0	64.5	207%
ST1			left		66.7		
ST2	129	2+520	axis	8.0	75.9	113.8	50%
ST3			right		64.5		
ST4			left		62.0		
ST5	157 (T1)	2+664	axis	10.10	81.5	134.4	65%
ST6			right		65.6		
ST7.2			left		81.7		
ST8.2	185	2+800	axis	12.5	87.7	136.7	56%
ST9.2			right		84.1		
ST10.2			left		92.3		
ST11.2	195 (T2)	2+850	axis	12.30	94.4	134	42%
ST12.2			right		90.7		
ST16			left		89.5		
ST17	239 (T3)	3+070	axis	14.20	102.2	147.4	44%
ST18			right		115.0		
ST19			left		122.9		
ST20	264 (T4)	3+195	axis	13.80	118.1	160.6	36%
ST21			right		94.3		
ST22			left		57.9		
ST23	283	3+290	axis	10.50	53.5	124	132%
ST24			right		54.6		
ST25			left		27.4		
ST26	301	3+380	axis	6.30	29.3	81.5	178%
ST27			right		25.2		

Discrepancies in geotechnical projects are customarily and vaguely explained with soil heterogeneity and uncertainty caused by not having a borehole close to the cross section where the calculation is made. This would be relevant if both underpredictions and overpredictions were observed. Herein, the systematic overprediction precludes that heterogeneity and uncertainly are the main culprits by themselves. One reason already mentioned –related to heterogeneity– for a systematic overprediction is the choice to “err on the clay side” and ignore occurrences of small-thickness sandy layers within the fined-grained layers I,II, IV and V in the design soil profile. The fact that the two shortest embankment cross sections have the largest overpredictions draws attention to the soils closer to the surface, which are responsible for the highest percentage of settlement. Although the depositional history of the site justifies a normally consolidated soil, as was assumed to be the case for surficial layer I, aging can produce an apparent preconsolidation pressure, also mentioned by Karstunen et al. (2020) as cause of overpredicting settlements of soft clays (if ignored). Fluctuations of the water table may have also created a surficial crust that is not accounted for in the design soil profile. A final hypothesis explaining the lower displacements measured for the short embankment sections is related to potential sparse lenses of lightly expansive clays, responsible for swelling stresses measured in laboratory tests in the range of 10-55 kPa. There may be a downward displacement of the low embankments, when the foundation clays undergo temporary desiccation. During the rainy season, the desiccated clay may partly swell and the settlement effect is reversed.

The second issue is what differences between observed and predicted settlements are recorded in actual projects. Without published numbers from real projects, the next best option is large-scale instrumented tests combined with prediction competitions, which is the approach Karstunen et al. (2020) followed in their class project. According to the data they used, predictions by professionals varied from 61% underprediction to 69% overprediction.

5 Concluding remarks

This paper selected aspects of a motorway embankment case study that are suitable for instruction even in an introductory soil mechanics course, either for a lecture presentation, or for an assignment, or for both. In the context of an assignment with given soil profile and design parameters, students can readily calculate one-dimensional consolidation settlements for a succession of silt and clay layers separated by sand and gravel layers and compare them with measured values. Then, the instructor can comment on the good comparison of the numbers calculated by the students and those in the project report. The large magnitude of measured and calculated settlement is an opportunity to connect geology and resulting soil profile and parameters. It is fortuitous that the project location is very close to the Thermopylae Pass (the place of the battle between Persians and Ancient Greeks) known world-wide for its morphology (pass), which has changed beyond recognition in the last 2500 years due to the same sedimentation processes that created the deep soil deposits underneath the motorway embankment. The instructor can also show to the students comparisons of measured settlement to settlement predictions made by other students and by practitioners. The comparison of calculated and measured settlements is an opportunity to introduce the students to the variability expected in real projects and dispel the notion of the geotechnical engineer attempting to calculate the one correct value, as many students are led to believe by solving problems with the specific method taught in class and the specific parameters given in the problems.

What is more, this case study raises questions suitable for discussion beyond the classroom. Two such types of questions are identified. One question concerns the chosen calculation method. The systematic overprediction of the settlements suggests that a systematic element of soil behavior may have been misrepresented, e.g. the potential of the shallow clay layers exhibiting an overconsolidated-type behavior. A second question –of much broader relevance– concerns the expected variation between calculated and measured values in practice. Published ranges of differences between calculations and measurements are typically obtained from prediction competitions where practitioners and researchers are invited to predict the behavior (e.g. settlement or ultimate bearing load) of a testing facility. These results may not be representative of the ranges expected in real projects, where the stakes are higher than in competitions. This observation underscores the value of practitioners publishing results comparing calculations and measurements from real projects.

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