Let’s Bring into the Classroom the Reality of Estimating Soil-Engineering Properties

A.D. Platis[1], V.E. Dimitriadi[2] & K.T. Malliou[1]

[1]*Geoconsult Ltd., Athens, Greece*
aplatis@geoconsult.gr, komalliou@geoconsult.gr

[2]*Edinburgh Napier University, Scotland, UK*
V.Dimitriadi@napier.ac.uk

ABSTRACT: Understanding the geotechnical conditions at a project site is of fundamental importance in making informed decisions and shaping optimum engineering solutions. This knowledge mainly covers the determination of the soil stratigraphy and the evaluation of a representative set of design soil properties. While the first is typically explored through the drilling of boreholes and is performed on site, the second aspect is the outcome of in-situ observations and tests and laboratory work, which typically reveals the high variability of the soil. This soil variability however is not normally perceived by young geotechnical engineering graduates, who are used to dealing with uniform soil layers in terms of composition and behaviour during their academic studies. This often leads to frustration and poor performance in their initial professional steps. To that end, the present paper has the ambition to shed some light into the intricate task of soil characterisation and bring into the classroom tangible examples of how the soil non-uniformity is tackled in the professional arena, using as an example the soil properties pertinent to consolidation settlement.

Keywords: site investigation, laboratory testing, geotechnical education, educational material

1 Introduction

A key component in geotechnical practice builds upon evaluating the design parameters of the soil profile at hand, which is based on the characterisation of the encountered layers, acquisition of soil samples and execution of laboratory testing. The specific geotechnical engineering attribute is eloquently communicated in the 1st John Burland Lecture by Atkinson (2016), who stated that ‘first-degree graduates with a geological map and memoir, some tubes of soil from the site, a pencil and a paper, should – at a minimum – be able to produce safe and serviceable designs for simple foundations and slopes’. More specifically, Atkinson (2016) describes a suite of three basic tasks that young geotechnical engineers should be able to deliver, namely: (i) model the ground, (ii) evaluate the design parameters and (iii) design simple slopes and foundations. Regarding the design of simple slopes and foundations, it is estimated that most geotechnical graduates will be sufficiently proficient, as opposed to evaluating design soil parameters. The reason behind this is that soil parameters are typically the input information in most worked examples and exercises, which are found in geotechnical engineering textbooks. As a result, students have limited opportunities to practice their skills in determining design soil properties.

Given the above, most young geotechnical engineers generally lack the skills to determine design soil properties and do not perform adequately in related tasks. In many cases, they are even astonished to discover that real soil conditions exhibit a high degree of variability, which needs to be reflected into a unique set of soil property values. In the above context, the question addressed in the present paper concerns the inclusion of actual site investigation and laboratory data –which are normally used for the estimation of design soil properties– in geotechnical education.

To achieve this objective, the first section of the paper compiles and presents information from the site investigation and in situ and laboratory testing from two actual projects in Greece. The first project is located in the island of Corfu (Geoconsult, 1999b) and the second in Nea Karvali, Kavala (Geoconsult, 1999a), as indicated in Figure 1. The criteria for selecting the two projects were (i) the uniformity of the encountered soil profile, (ii) the availability of field and laboratory data and (iii) the availability of during and/or post-construction monitoring data. Monitoring data are particularly important, as they allow the back-calculation of some key soil properties for both projects, as well as a comparison between the back-calculated values and the ones estimated from the in situ and laboratory tests. This comparison is further extended to include typical values from the literature aiming to provide a more complete overview of the possible range of variation of the soil properties under examination. This paper focuses exclusively on the soil properties pertinent to consolidation settlement and their evolution.

The purpose of the above task is to (i) familiarise students with the soil properties that are typically required in the design of different projects and how those are determined, (ii) highlight the potential and (expected) scatter in the obtained values and (iii) showcase how the obtained in situ and laboratory values compare against the 'real' values (based on the back-analysis results) and 'typical' values (based on the literature). Going a step further, the paper also explores the potential development of suitable educational material, which can be incorporated in curriculum design, so that students can experience and appraise the practical aspects of their academic education.

Figure 1. The locations of the selected projects

2 Extension of the apron area of Corfu Airport

2.1 General information of the project

The location of the site is at the South part of the Corfu Airport 'Ioannis Kapodistrias' in the town of Corfu. The project area was flat and swampy with the lagoon water depth ranging between 0.05 m to 0.30 m. The main construction activities planned in the area of the project included:

- An extension towards the South of the apron area with dimensions 325.5 m×200 m.
- Ground supplies area with dimensions 40 m×162 m.
- New connecting taxiway of total length of 623 m at the South-West end of the extension of the apron.
- A 12 m wide service road, parallel to the East boundary of the extension of the parking area.

2.2 Site investigation and laboratory tests programme

The site investigation programme for the airport extension included 4 exploratory boreholes of depth equal to 15.22-24.30 m, and 18 CPT of depth 6.00-24.30 m in the area of the planned extension of the apron. CPTs provided information on the cone resistance (q_c – MPa), local skin friction (f_s – MPa), friction ratio (FR = f_s/q_c – %), soil type and generated excess pore pressure (u – MPa, in the tests with piezocone). Based on the retrieved soil samples a series of laboratory tests were carried out. The type of tests was adjusted to the nature of the soil, the method of sampling and the scope of the investigation. Classification tests as well as physical and mechanical properties tests were carried out on typical samples from the boreholes. The following laboratory tests were performed, with the number of them included in parentheses: determination of water content (22), specific gravity (22), unit weight (14), Atterberg limits (22), sieve analyses (22), hydrometer tests (22), oedometer tests (11), unconfined compression tests (7) and unconsolidated undrained (UU) triaxial compression tests (15).

Based on the results of the geotechnical investigation it was possible to trace the soil stratigraphy in the area of the planned extension. A simplified yet representative design soil profile is presented in Figure 2 and a brief description of each soil layer is provided below:

- Layer [1]: Very soft to soft CLAY of medium to high plasticity (CL-CH), turning deeper to firm and then to stiff clay. The thickness of this layer was highly variable, ranging between 2-17 m.
- Layer [2]: A layer of loose to medium dense SAND and SILT (SM) to organic SILT (OL) (layer [1b] of the original geotechnical investigation), which was at some places encountered as a lens within layer [1] and at other places between layers [1] and [3].
- Layer [3]: Stiff to very stiff CLAY of medium plasticity (CL) (layer [1a] of the original geotechnical investigation).
- Layer [4]: Very stiff to hard CLAY of medium plasticity (CL) (MARLY BEDROCK).

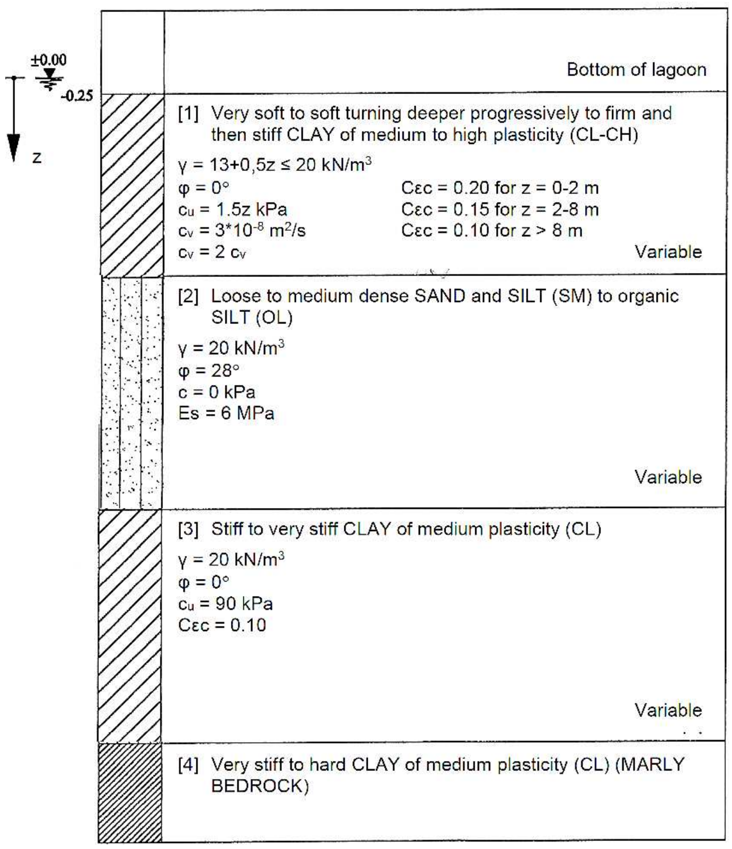

Figure 2. Design soil profile at the Corfu International Airport

2.3 Selection of design properties

The evaluation of the data obtained from the site investigation led to the determination of the design soil parameters, which are included in Figure 2. Relevant graphs and tables of the geotechnical investigation are available as EXCEL files (see Corfu Data, Karvali Data) in http://www.geoconsult.gr/en/publications/. As already mentioned, the focus herein is on the soil properties that most affected the design process concerning consolidation settlement. Due to the variability in thickness (3-17 m) and in consistency (very soft to soft) of clay layer [1], it was concluded that it would be the controlling layer in the geotechnical design of the project. Its compressibility was expected to lead to –uneven– large settlements, which would develop slowly. Moreover, the development of settlements was expected to be spatially affected by the intermittent presence of the intermediate drainage layer (layer [2]).

For the calculation of the expected settlements, the Compression Index (Cc) was estimated from oedometer test results and was found to vary between 0.134 and 0.879 with an average value of 0.359. The Compression Ratio (CR), designated as Cεc in Figure 2, was consequently computed as the ratio of Cc/(1+e_o), where e_o denotes the initial void ratio. This computation provided the variation of CR with depth, which was approximated with the following design values of CR (see also Figure 3):

- CR = 0.20 for z = 0-2 m
- CR = 0.15 for z = 2-8 m
- CR = 0.10 for z > 8 m, (z counting from ±0.00 m)

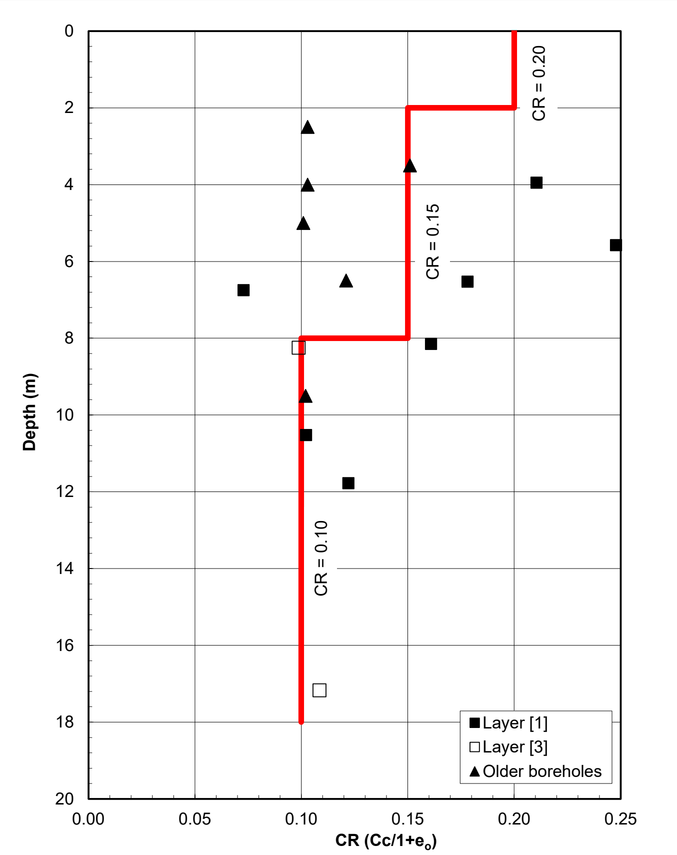

Figure 3. Distribution of CR with depth and design curve (in red)

To obtain a better understanding of the compressibility potential of layers [1] and [3], the compression index Cc values obtained from oedometer laboratory tests are compared with typical values from the literature, which are summarized in Table 1. Using the values in Table 1 and an average project value of LL equal to LL = 42%, gives the Cc values plotted in Figure 4. It is observed that the measured values of Cc in the laboratory (Cc = 0.13 – 0.88, with an average value Cc = 0.36) compare well with the values proposed by literature. What is important to note from Figure 4 is the high variability of the reported values in the literature, which come from different clays around the world, and highlight the need of site-specific laboratory tests.

2.4 The design challenge: Consolidation and settlement acceleration methods

From the relevant geotechnical calculations (Geoconsult, 1999b) the expected primary consolidation settlements for embankments bearing directly on the bottom of the lagoon were estimated between 600-1100 mm. Moreover, the time required for the completion of consolidation was estimated to be of the order of several decades. The expected settlement magnitude along with the non-uniform accumulation pattern and the required consolidation time would pose a significant hazard upon the structural integrity of the planned works. Given the above challenges, it was decided to adopt a ground improvement method, which would accelerate consolidation and would lead to the completion of most of the expected primary consolidation settlements prior to the construction of the pavement. In addition to that, the ground improvement method should also reduce the anticipated long-term secondary compression settlements due to the high plasticity and high organic content of the soft clay layer.

The adopted soil improvement method consisted of preloading in combination with the installation of stone columns, which would act both as vertical drains, for the acceleration of pore pressure dissipation, and as reinforcing elements. The stone columns were 0.70 m in diameter and extended within the upper very soft to soft clayey layer [1] to a depth ranging between 6.00-15.00 m. Their axial distance varied between 2.00-4.50 m. Preloading was performed with an embankment at least 1.50 m higher than the final pavement level, in order to reduce long term secondary compression settlements and further accelerate the completion of expected settlements.

Table 1. Indicative values of compression index Cc

Type of soil	Cc	Reference
CL soft clay	0.34	*Kaufmann and Shermann (1964), Louisiana clays, USA*
CH clay of high plasticity	0.84	
CH soft clay with silt layers	0.52	
New York clays	Cc = 0.009*(LL – 10)	*Terzaghi and Peck (1967)*
Clays	0.1 – 0.8	*Budhu (2011)*
All clays	Cc = 0.01*(LL-13)	*Ameratunga et al. (2016) [from USACE (1990)]*
Clays from Greece and parts of US	Cc = 0.4*(e_o-0.25)	*Ameratunga et al. (2016) [from Azzouz et al. (1976)]*
Medium to high plasticity Maroussi clay (CL2-CH)	0.13 (SD = 0.05)	*Tolis et al. (2006)*
Clays from Greece (mostly CL and a few CH)	0.04 – 0.33	*Bardanis and Kavounidis (2001)*

2.5 Comparison of predicted vs observed performance and back-analysis

Following the installation of the stone columns and the construction of the preloading embankment, the area of the project was systematically monitored over a period of 62 months (12/09/02 to 17/10/07). From the monitored area, two locations (denoted herein as A and B) presented the greatest interest regarding the development of the primary consolidation settlements. Location A was in the area of taxiway D, where the thickness of layer [1] ranged between 7.00 – 12.00 m (7.00 – 9.00 m at the location of the settlement monitoring, hence the considered values in the back-analysis) and the stone columns were arranged in a 3.00 x 3.00 m grid. Location B was in the area of an initial trial embankment, where the thickness of layer [1] ranged between 11.00-19.00 m (17.00 m at the location of the settlement monitoring, hence the considered value in the back-analysis) and the stone columns were arranged in a 2.50 x 2.50 m grid (for more details refer to Platis et al., 2010).

Note that the back-analysis of the time evolution of settlements is not presented in the present paper, as it was a very extensive and multi-parametric process, involving not only the coefficient of compressibility c_v, but also the secondary compression rate C_α, and the horizontal coefficient of consolidation c_h. It was therefore decided by the authors to omit this part of the back-analysis, so that the ultimately available educational material is suitable for use in the classroom. Using the data from the monitoring equipment (settlement plates and electric piezometers), back analyses were executed, including, among others, a reassessment of the compression ratio of the clayey layer [1], which mainly contributed to the measured primary consolidation settlements. According to the back analysis, a fluctuation in the obtained CR values was observed. Namely, for Location A, CR ranged between CR = 0.19÷0.21 and for Location B, the obtained value was significantly lower and equal to CR = 0.09.

Due to the above difference in the compressibility of the clayey layer [1] between Locations A and B, the results of the available consolidation laboratory tests were reviewed. This confirmed a differentiation of the layer's compressibility, which increased from the eastern towards the western part of the project area. Therefore, the initial design was re-visited, and settlements were re-evaluated considering the new average values of the CR, i.e. average $CR_{[1]}$ = 0.20 for location A and average $CR_{[1]}$ = 0.10 for location B. These new design values correspond to Compression Index values equal to Cc = 0.533-0.589 and

0.252, respectively, for an average initial void ratio e_o = 1.803 for location A and e_o = 1.52 for location B. The different values of the Compression Index Cc, namely lab test results, literature values and back-calculated values, are summarized in Figure 4, so that a more comprehensive overview is provided. The main observation from this plot is the significant scatter observed, both in the reported Cc values in the literature (plotted in grey rhombuses), as well as in the values obtained from the laboratory testing programme (plotted in red squares) and the back-calculated ones (in cyan circles). Namely, it is evident that soil formations with similar classification are not necessarily characterised by comparable values of soil properties. Even within the same project, it may be necessary to consider different design soil properties in order to capture the soil response under the same loading conditions. A solid proof of the above statement is the set of back-calculated values of Cc, which further indicate the highly variable nature of one single soil layer.

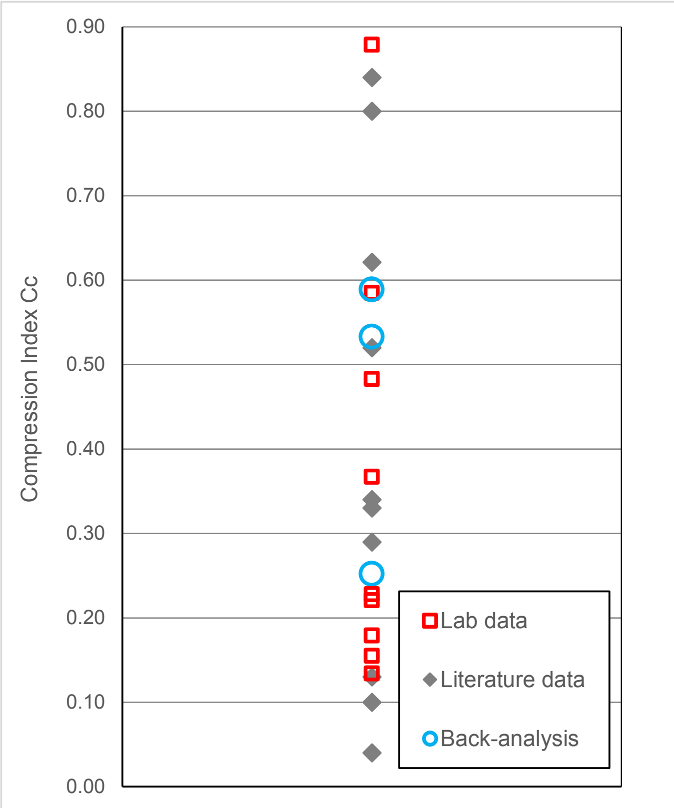

Figure 4. Compression Index values Cc as specified from lab and literature data and the back-analysis

3 Construction of an oil refinery in Nea Karvali, Kavala

3.1 General information on the project

The site selected for the construction of the Kavala Oil Products Terminal of MOTOR OIL HELLAS was located 3 km west of the village of Nea Karvali, in north-Eastern Greece towards the eastern end of Valtos Bay. The site was at a low elevation area (0.00 m to +1.00 m from Mean Sea Level), which had been extensively used as a sand borrow area. The project included the construction of four floating roof fuel tanks, four fixed roof fuel tanks, one water tank, loading gantries, office building and a warehouse, a power substation and a parking area, as well as the offshore construction of an oil unloading jetty. The fuel tanks would have a capacity of 1,000 - 3,000 m^3 and diameter between 12.50 - 22.00 m. The floating roof tanks would be supported by a concrete ring foundation, backfilled by coarse material. The fixed roof tank's steel bottom would rest on a layer of well compacted granular fill. The area of the fuel tanks would be surrounded by a concrete spillage containment wall.

3.2 Site investigation and laboratory tests programme

The site investigation programme included 3 off-shore and 4 on-land exploratory boreholes as well as 12 on-land CPT. Based on the retrieved soil samples a series of laboratory tests were carried out. More specifically, classification tests as well as physical and mechanical properties tests were carried out on typical samples from the boreholes. The following lab tests were executed, with the number of them included in parentheses: sieve analyses (64), hydrometer tests (39), determination of natural water content (64), Atterberg limits (64), specific gravity (39), unit weight (23), oedometer tests (14), unconfined compression tests (15), and unconsolidated undrained (UU) triaxial compression tests (16).

Based on the results of the geotechnical investigation the design soil profile is presented in Figure 5 and a brief description for each soil layer is provided below:

- Layer [1]: Medium dense to dense, fine to medium SAND (SP), turning to silty SAND (SM) with depth.
- Layer [2]: CLAY of medium to high plasticity (CL2-CH) very soft to soft, with organic SILT (OL)
- Layers [3] and [4]: CLAY to sandy CLAY of low to medium plasticity (CL-SC) with thin intercalations of SAND to silty SAND (SM-ML).
- Layer [5]: Coarse grained SAND (SP) to silty SAND, at depths below 24.60 m, in only in one borehole, hence, was not considered in the design soil profile (not shown in Figure 5).

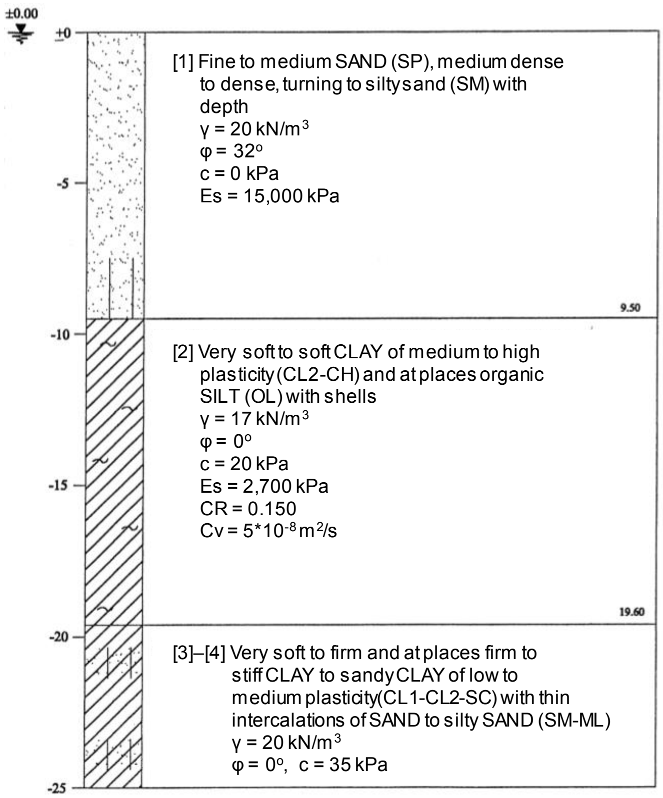

Figure 5. Design soil profile from Nea Karvali

3.3 Selection of design soil properties

Figure 5 also includes the design soil parameters, which were evaluated based on the site investigation programme. An overview of the obtained data from the field and laboratory tests is provided in: http://www.geoconsult.gr/en/publications/ (see Karvali Data). The selection of the Compression Index Cc and the Coefficient of Consolidation c_v is explained herein, as they are linked to the main identified design issues of the project area, and particularly to layer [2].

Regarding the Compression Index (Cc), the (oedometer) laboratory-obtained values range between 0.077 - 0.531, with an average value of Cc = 0.344. Typical values of Cc obtained from the literature are also summarized in Table 1. It is observed that the project values are lower than the range of values reported by Kaufmann and Shermann (1964) for high plasticity clays (Cc = 0.52 - 0.84). Also, the value of Cc estimated by the expression proposed by Terzaghi and Peck (1967), for an average project value of LL equal to LL = 42.7%, is Cc = 0.009*(42.7-10) = 0.29, which falls within the range of the measured Cc values in the laboratory. The Compression Ratio (CR = Cc/(1+e_o)) was estimated from the Cc and corresponding e_o values of each consolidation test and ranged between CR = 0.035 - 0.222.

Regarding the Coefficient of Consolidation c_v, the laboratory-obtained values for c_v range between $2.6*10^{-8}$ – $87.1*10^{-8}$ m^2/s, with an average value of $18.9*10^{-8}$ m^2/s. Typical values of c_v obtained in the literature are summarized in Table 2. The comparison between the design project values and the proposed values in the literature is better appraised in Figure 6. It is observed that on average the best agreement is achieved with the upper values reported by Van Tol et al. (1985) and Wallace and Otto (1964). It is also noted that most of the reported values in the literature are well below c_v = 20 * 10^{-8} m^2/s and therefore closer to the lower range of the design value of c_v = $2.6*10^{-8}$ m^2/s.

For the specific project, the final selection of the design values for CR and c_v was based mainly on the consolidation test results and partly on published literature; the respective design values were CR = 0.15 and c_v = $5*10^{-8}$ m^2/s.

Table 2. Typical values of the Coefficient of Consolidation c_v, proposed in the literature

Type of soil	c_v (m^2/s) $*10^{-8}$	Reference
Soft Blue clay (CL – CH)	1.6 - 26	*Wallace & Otto, 1964*
Organic Silt (OH)	5 - 170	*Lowe et al., 1964*
Chicago City Clay (CL)	8 - 11	*Terzaghi & Peck, 1967*
Sandy silty clay (ML – CL) dredge spoil	5 - 20	*Van Tol et al., 1985*
Organic Silts and Clays (OH)	1 - 10	*Sivakugan, 1990*
San Francisco Bay Mud (CL)	2 - 4	*Budhu, 2011*

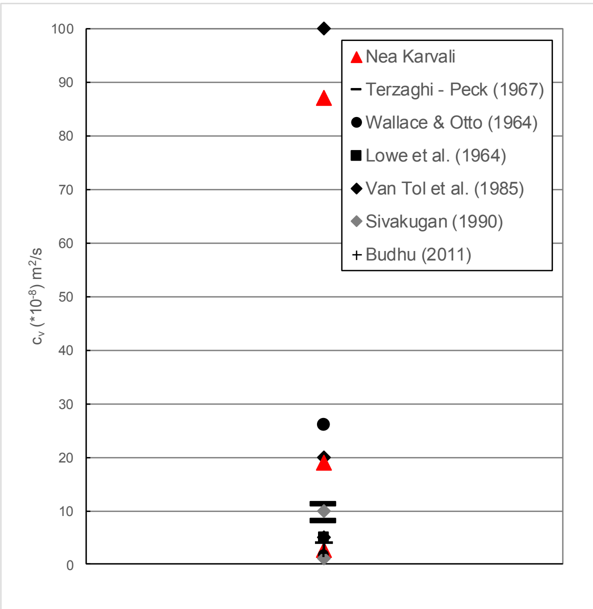

Figure 6. Comparison between design cv values and proposed ranges in the literature

3.4 The design challenge: Consolidation, preloading and prefabricated drains

The main design challenge in this project was the anticipated settlements of the tanks, which, due to their large diameter, also had a considerable depth of influence. According to the performed calculations, for an applied pressure of 100 kPa, settlements varied between 115-190 mm approximately, depending on the tank diameter, with the amount of the anticipated settlements being proportional to the diameter of the tanks. Furthermore, the compressible layer [2] was significantly affected and was expected to contribute the greatest portion of the tank settlements. Even more so, the combination of its considerable thickness (approximately 10 m) and the existence of a single drainage path (pore pressure dissipation would practically take place only through the overlying permeable sandy layer [1]), would mean that settlements were expected to develop slowly. The presence of fine sandy seams could potentially facilitate and accelerate drainage and settlement development, under the condition that they were continuous and extended outside the perimeter of the tanks.

This long-term settlement problem in the tank farm area was proposed to be overcome with preloading and by accelerating pore water pressure dissipation with vertical strip drains. Preloading could be achieved either by constructing an embankment or by the lowering of the water table in the sand or a combination of the two. Following a parametric analysis of 5 triangular drain spacings between 1.50-2.50 m, for selecting the optimum drain arrangement, a 1.50 m equal sided triangular grid was selected based on technical and economical considerations. This solution was combined with the construction of two preloading embankments, one 6.00 m high in the tank farm area and one 7.50 m high in the water tank area, and with a 4.00 m draw-down of the GWL by pumping from 16 water wells surrounding the preloaded areas. The ultimate target of this solution was the completion of primary consolidation within 6 months.

3.5 Comparison of predicted vs observed performance and back-analysis

After the construction of the preloading embankments, monitoring was carried out both for the tank farm area (10 settlement plates (S1-S10)), and the water tank area (2 settlement plates (S11-S12)). The monitoring data from the tank farm area are presented in Figure 7, which include measurements of settlement with time, carried out over a period of 200 days. Location ST-3 refers to the centre of tank TK-3 (S for settlement). These locations were selected (a) because they were close to borehole locations (more accurate soil profile) and (b) because they were situated at different geometrically locations of the preloading embankment. Based on the presented measurements the following observations are made:

- Approximately 7 months after the construction of the embankments, the total settlement was of the order of 461-591 mm (average settlement in the order of 516 mm).
- The rate of settlement was generally faster than expected, which is a common finding in the field (Viggiani, 2019).
- A slight increase in the rate of settlement was observed after approximately 95 days, i.e. at the end of January 2002. This was attributed to an increase of the weight of the embankment due to the unusually high precipitation and snowfall in January.
- The final rate of settlement ranged between 6-10 mm/month, which was within the usual limits given in the literature (of the order of 5-15 mm/month) for terminating preloading.

According to the ground improvement design (Geoconsult Ltd., 2000), the expected consolidation settlement after 7 months due to the preloading embankment in the tank farm area was between 514-534 mm. This range was very comparable to the observed amount of settlement (during the monitoring period), hence no re-evaluation of the Compression Ratio (CR) was performed. On the contrary, the rate of settlement was slightly faster than expected, therefore a back analysis of the coefficient of consolidation c_v was carried out. The results of this back analysis showed that a value of $c_v = 6*10^{-8}$ m^2/s gave a better fit to the actual monitoring data (see Figure 7), which is 20% higher than the design value ($c_v = 5*10^{-8}$ m^2/s).

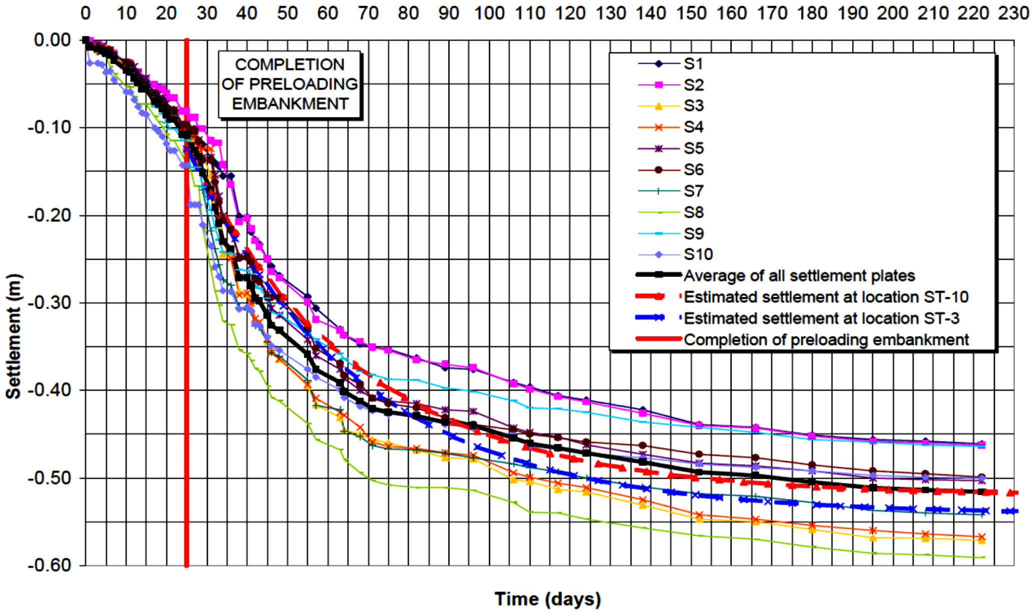

Figure 7. Preloading settlement data from 10 settlement plates as compared to design curves with c_v = $6*10^{-8}$ m^2/s

4. Development of educational material and future work

The previously presented material is intended to provide a comprehensive set of geotechnical data from real projects and form the basis for the development of educational material, which can be used in geoengineering education. This project-based learning experience is built around three components, which very much resemble the different stages of a geotechnical project, namely identifying the design soil profile, determining the design soil properties and performing fundamental geotechnical calculations. In the remaining of this section, these three components - each targeting different educational needs - are described, links to the existing material are established and indicative learning outcomes identified. Furthermore, future work considerations are suggested.

The first component focuses on exploring the subsurface conditions and composing the design soil profile. For this purpose, the principles of designing and executing a site investigation and laboratory testing plan need to be explained, in addition to developing skills in the interpretation of the obtained borehole data. In both projects the design soil profiles are already provided (Figures 2 and 5), so the authors propose indicative teaching activities. Namely, relevant background knowledge can be provided in the classroom in the form of technical guidelines as part of a relevant geotechnical engineering module. Also, provided that factual borehole data are available, students can compose their own design soil profile. The purpose of these activities would be to cover the following learning outcomes:

- Evaluating in advance the depth of influence of the planned structures, depending on their geometry and loads, in order to select the desirable depth of the boreholes.
- Considering an appropriate borehole arrangement in order to cover most significant structures (in terms of their magnitudes of loads and their sensitivity in settlements) and get sensible geotechnical sections afterwards, in order to decide the minimum required number of boreholes.
- Conducting a desk study in order to obtain published geological and geotechnical information for the area of the project and deciding the most effective method of drilling and sampling (e.g. rotary drilling with undisturbed samples in soft clays, SPT sampling in sands, wagon drilling or geophysical survey in karst terrain, CPT in deep soft/loose deposits, trial pits and large diameter sampling in filled or dump areas, etc.).
- Preparing a laboratory-testing programme (types of tests carried out depending on the soil type).
- Interpreting borehole data.
- Producing typical geological cross-sections.

The second component is oriented towards interpreting lab and field test data to determine relevant design soil properties prior to any geotechnical design. To this end, the available batch of information from both projects is available to download as both raw lab data, but also organised in tables and graphs showing the range of variation of the soil properties and their distribution with depth. Using the provided material, learning activities should aim in mobilising the students to use the site investigation data and embed the Eurocode 7 (EC-7) framework regarding the selection of 'characteristic' and 'design' soil properties. A comparison can be made between the set of the design values, which were adopted in the projects and the students' choices. The proposed activity targets the following learning outcomes:

- Interpreting field and lab test results
- Appraising the principles of EC-7 in defining design soil properties.

The third component of the proposed educational intervention aligns with the final stage of any project in geotechnical engineering, namely that of design. In this section, students should be given the opportunity to act as design engineers, use the design soil profiles and the selected set of soil properties of the previous stages to shape suitable engineering solutions. It is believed that this activity is better to be limited to the design of a specific structure, such as a shallow foundation (i.e. calculation of settlements, bearing capacity and time required for completion of consolidation when a clay layer is present) so that the various design challenges are covered in depth. With reference to the presented projects, consolidation settlements and consolidation time were the main geotechnical challenges and indicative results are reported considering the mean values of the obtained soil properties. In that context, students can perform their own calculations and explore the sensitivity of their results against selected soil properties. For comparison purposes, the detailed calculations for both projects are available upon communication with the authors. The purpose of this final activity will be to cover the following learning outcomes:

- Calculate settlements and consolidation time for indicative structures
- Investigate the sensitivity of the obtained results against the fluctuation of the selected soil properties (e.g. the change in settlements by adopting higher or lower values of Cc, or thickness of the compressible layer, groundwater table fluctuations)
- Explore ways of improving the original design (i.e. means of accelerating consolidation) and potentially performing a preliminary cost-benefit evaluation.

This last component has multiple benefits as it provides students an opportunity to critically reflect on the significance of the selected soil properties upon the design of civil engineering projects both on qualitative and quantitative terms.

5. Conclusions

In the present paper, the authors have attempted to touch upon the intricate issue of selecting design soil properties, based on the available site investigation reports and lab test data and explore ways of introducing this aspect into the classroom. For that purpose, two projects from Greece were selected, representing the two extremes that one may encounter in practice: one with very uniform layering and soil properties (Nea Karvali) and one with pronounced variability both in stratigraphy and soil properties (Corfu). A database of geotechnical data has been created from these two projects, which can be used as a basis for the development of additional learning activities. The data from the Nea Karvali project could be used for simple soil mechanics calculations, whereas the data from Corfu can be used for more complex soil mechanics calculations, such as sensitivity analyses in terms of strata thickness or soil properties variability, probability of bearing capacity failure estimations, differential settlement estimations, etc.

The authors would like to invite educators in geotechnical engineering to include this material in their classrooms and provide feedback and comments for its further improvement. Additionally, practicing engineers can contribute in extending this database with soil formations from other countries.

Acknowledgements

The authors would like to express their gratitude to Marina Pantazidou for her encouragement for the preparation of this paper and the reviewers and editors of the GEE 2020 Proceedings for the valuable comments and feedback.

References

Ameratunga, J., Sivakugan, N., Das, B.M., (2016). Correlations of soil and rock properties in geotechnical engineering, Springer India, New Delhi, India.

Atkinson, J.A. (2016). Basic geotechnical engineering skills: What can graduates do? 1st John Burland Honor Lecture, ISSMGE Int. Conf. 'Shaping the Future of Geotechnical Education' SFGE 2016, Belo Horizonte, Brazil, Oct. 20, https://www.issmge.org/education/videotaped-honor-lectures/first-john-burland-lecture, (accessed February 25, 2020).

Azzouz, A.S., Krizek, R.J., Corotis, R.B. (1976). Regression analysis of soil compressibility. Soils and Foundations 16(2), pp. 19-29.

Bardanis, M., Kavounidis, S. (2001). Comparison between Compression Index from Empirical Relations and Tests on Clayey Soils from Greece, 4th Hellenic Conference on Geotechnical and Geoenvironmental Engineering, Athens, Greece, 1, pp. 131-138 (in Greek).

Budhu, M. (2011). Soil Mechanics and Foundations, 3rd ed (1st ed. 2000), Wiley, New York, USA.

Geoconsult Ltd. (1999a). Kavala Oil Products Terminal – Geotechnical Investigation. Motor Oil Hellas.

Geoconsult Ltd. (1999b). Extension of Corfu Airport Apron Area. Ministry of Public Works (in Greek).

Geoconsult Ltd. (2000). Kavala Oil Products Terminal – Design of Ground Improvement by Preloading. Motor Oil Hellas.

Kaufman R.I., Sherman, W.C. (1964). Engineering measurements for the Port Allen lock. Journal of the Soil Mechanics and Foundations Division 90(5) pp. 221–248.

Lowe, J., Jaccheo, P.F., Feldman, H.S. (1964). Consolidation testing with back pressure. Journal of Soil Mechanics and Foundations Division, ASCE, 90(SM5), pp. 69-86.

Platis, A.D., Michalis, E.K., Malliou, K.T. (2010). Soil Improvement by Preloading Accelerated by Stone Columns – The Case of Corfu Airport, 6th Hellenic Conference on Geotechnical and Geoenvironmental Engineering, Volos, Greece, 1, pp. 441-448 (in Greek).

Sivakugan, N. (1990). Development of marshy areas in Colombo, Sri Lanka. Proceedings of the 10th South Asian Geotechnical Conference Taipei, 1, pp. 469-472.

Terzaghi K., Peck, R. (1967). Soil mechanics in engineering practice, 2nd ed, Wiley, New York, USA.

Tolis, S.V., Kavounidis, S., Bardanis, M.E. (2006). Geotechnical Characteristics of the Maroussi Clay, 5th Hellenic Conference on Geotechnical and Geoenvironmental Engineering, Xanthi, Greece, 1, pp. 293-300 (in Greek).

USACE (1990). Engineering and design – settlement analysis, EM1110-1-1904, Department of the Army, US Army Corps of Engineers.

Van Tol, A.F., Brassinga, H.E., Elprama, R., Van Rijt, C. (1985). Consolidation of soft fine-grained dredged material. Proceedings of the 11th International Conference on Soil Mechanics and Foundation Engineering, USA, 2, pp. 687 – 690.

Viggiani, C. (2019), Porto Tolle test embankment - A full scale experiment on the consolidation of a thick clay layer, TC306 case-study webinar, https://www.youtube.com/watch?v=hWpyswdglBk (accessed February 25, 2020).

Wallace, G.B., Otto, W.C. (1964). Differential settlement at Selfridge Air Force Base. Journal of Soil Mechanics and Foundations Division, ASCE, 90(SM5), pp. 197-220.

Authors' bios

Athanasios Platis, Geoconsult Ltd, Greece

Athanasios Platis is the Managing Director and Senior Partner of the geotechnical consulting company Geoconsult Ltd since 1992. He obtained a diploma in civil engineering (5-yr degree) from the University of Patras and a M. Eng in Geotechnical Engineering from the University of California at Berkeley. Over the past 33 years he has been involved in most aspects of design and supervision of geotechnical engineering in Greece, the coordination of geotechnical investigations, geotechnical design and consulting, supervision design review and expert advice for various projects, such as harbours and port facilities, renewable energy, highway and railroad projects. He has co-authored many technical notes and conference and journal papers on a variety of topics, as a result of his extensive professional experience. He has a deep interest in the development of innovative resources aiming at training new geotechnical engineers and combining comprehensive theoretical background with practical aspects.

Vasiliki Dimitriadi, Edinburgh Napier University, Scotland, UK

Dr. Vasiliki Dimitriadi is a Lecturer (Assistant Professor) at Edinburgh Napier University, Scotland, UK, who specialises in the field of Geotechnical Engineering. She graduated in civil engineering from National Technical University of Athens (NTUA) (5-yr degree), obtained a M.Sc. degree from University of California Berkeley, USA, and a Ph.D. degree from NTUA. Between 2014 and 2018 she worked as a geotechnical consultant in Athens, Greece, and has obtained significant experience through her involvement in a wide spectrum of civil engineering projects. Most of her research experience focuses on earthquake-induced liquefaction and the response of shallow foundations in a liquefaction regime. Since joining Edinburgh Napier University in 2018, she has further expanded her research interests in ground improvement methods for foundation remediation under static conditions. She is also actively pursuing educational research opportunities, with a focus on project-based learning and is interested in creating opportunities for academia – industry collaborations.

Konstantina Malliou, Geoconsult Ltd, Greece

Konstantina Malliou is a member of the permanent scientific staff of the geotechnical consulting company Geoconsult Ltd and partner since 2013. She graduated in civil engineering from the National Technical University of Athens (NTUA) (5-yr degree). During her employment at Geoconsult Ltd she has been involved in a variety of projects and obtained key qualifications in the interpretation of geotechnical data and the subsequent geotechnical design of various projects, such as power plants, power distribution units, harbours and port facilities, industrial buildings, sanitary landfills, soil improvement. She has been a co-author in several publications in technical and conference papers and is particularly interested in joint initiatives between industry and academia that promote project-based learning.