# Highlighting Links among Geology, Index Properties and Mechanical Behaviour at the Beginning of a First Course in Soil Mechanics

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ABSTRACT: The paper reports the experience of the authors teaching Soil Mechanics to undergraduate students. The focus is on the first three or four weeks of the semester. The practice consists of establishing, for the main soil archetypes (sedimentary sands and clays and residual soils), a strong relationship between: i) geological conditions prevailing during soil formation and thereafter; ii) soil physical-identification parameters; iii) basic trends of mechanical soil behaviour. The purpose is to explain – through simple mental models – how to interpret the basic physical and geological characterisation of the soil, in order to anticipate the main difficulties for a given (simple) project. These difficulties may include insufficient bearing capacity, very large and delayed settlements (soft clay), large settlements under seismic loading (loose sands), etc. In light of these difficulties, some solutions (just the main idea) are discussed (soil reinforcement, acceleration of settlements, vibro compaction, deep foundations instead of shallow foundations, etc.). The experience of transmitting this perspective is that these early classes enable: i) a better apprehension of the subsequent truly mechanical chapters; ii) a proper evaluation by the students of the technical and practical relevance of the subjects; iii) a strong motivation for the study of the discipline.

*Keywords: Soil physical characterization, Geological conditions, Trends of mechanical behaviour*

## 1 Introduction

The traditional Soil Mechanics syllabus in a Civil (or Mining) Engineering degree course starts with an introductory section (with one or more chapters, according to the structure of the adopted textbook) on the soil physical parameters, as well as certain chemical-mineralogical features of clays. The physical parameters comprise the basic indices that express the proportion of the weight and volume of the three phases of the soil (water content, void ratio, porosity, degree of saturation), the various unit weights (total, dry, buoyant and solid particle) and also the identification characteristics: the particle size distribution curve and the Atterberg limits. These identification characteristics are the basis for the application of the Unified Soil Classification (Casagrande, 1948).

Then, the effective stress principle is introduced, followed by the chapters that deal with the soil strength and stiffness under various types of loading (confined, isotropic, triaxial in compression or extension, simple shear, drained and undrained). For non-saturated soils, the stresses in the three phases of the soil are explained to take into account the suction effect, and the behaviour under suction controlled conditions is discussed for the various types of loading. These aspects are covered in two ways, which correspond to the so-called Classical and Critical State approaches.

In most textbooks, the treatment of the physical parameters is essentially presented with reference to their laboratory determination, without a clear intention to establish a strong connection to the soil formation process in Nature and to the geological scenario prevailing at that time and site. Similarly, in those introductory chapters most textbooks lack the intent to explain how the interpretation of this set of physical parameters enables to anticipate some trends of the soil mechanical behaviour.

In our opinion, such options are going to limit the students' understanding of the future chapters on soil mechanical characterization due to the absence of awareness for what determines, in concrete cases, the higher or lower strength/stiffness of a particular soil. This point is more pertinent to the classical approach than to Critical State Soil Mechanics, but it applies to both.

Based on the physical characterisation of the constituent soils of the various layers and the geological context/scenario of the site, an experienced geotechnical engineer is often able to anticipate the essential problems that the soil mass presents and, consequently, is capable to take a number of major design decisions for a given project. The specific (quantitative) aspects of design naturally require the experimental determination of mechanical (and sometimes hydraulic) parameters, with knowledge and observance of the theoretical fundaments of Soil Mechanics.

It seems to the authors pertinent to raise the following question: is it possible to train students to anticipate the essential features of the mechanical behaviour of the ground mass, particularly those more unfavourable, based on the interpretation of soil physical characterisation data and of the site geological context? For clarity, one must delimit the context of the question: we are considering projects that involve ground with horizontal surface, to be loaded by civil engineering structures such as tanks, silos, embankments for transport infrastructures or for large industrial-logistic areas, or foundations of current structures. Complex geotechnical works are excluded, such as stabilisation of natural slopes, deep excavations and others.

This is the object of this paper, based on the experience of the authors in teaching Soil Mechanics to undergraduate students at the University of Porto. This experience allows to answer affirmatively to the question formulated above, as will be explained herein.

## 2 The current approach for the treatment of physical parameters

The approach adopted in the first chapters of most courses and textbooks is essentially focused on the characterization of physical parameters (with special laboratory emphasis) and omits, or gives insufficient emphasis, to the following essential questions for sedimentary soils:

i) what controls or characterises the physical state of the soil shortly after sedimentation?

ii) which physical parameters can be assigned to the soil shortly after sedimentation?

iii) which natural processes act mechanically (i.e., exert loading) on the soil following sedimentation?

iv) what relation do these processes and the physical state of the soil have with the geological scenario/context, in particular with the age of the sedimentary deposit?

v) what is the effect of these processes on the soil physical parameters?

vi) how does the alteration of the physical parameters influence, in qualitative terms, the mechanical response of the soils when loaded by simple Civil Engineering structures?

These questions are now discussed for the two sedimentary soil archetypes: sands and clays.

## 3 Sandy soils

Figure 1 schematically shows that the grain size distribution determines the soil void ratio interval $e_{max}$ - $e_{min}$. However, it is rare to find in textbooks an additional explanation concerning the following items, which are essential for starting to understand the mechanical behaviour of granular sedimentary soils:

i) at the "moment" of sedimentation each soil assumes its maximum void ratio, $e_{max}$;

ii) due to natural loading (weight of new sediments, earthquakes, etc.), the *in situ* void ratio moves progressively away from $e_{max}$ and tends to $e_{min}$;

iii) the reduction of void ratio occurs due to particle rearrangement, with progressive elimination of unstable equilibrium situations, initially very numerous;

iv) this structural alteration remains essentially preserved, even when Nature removes by erosion the overlying layers that caused that evolution;

v) the reduction of void ratio, expressed by an increasing density index, $I_D$, has a clear mechanical consequence, increasing stiffness (and strength) of the soil.

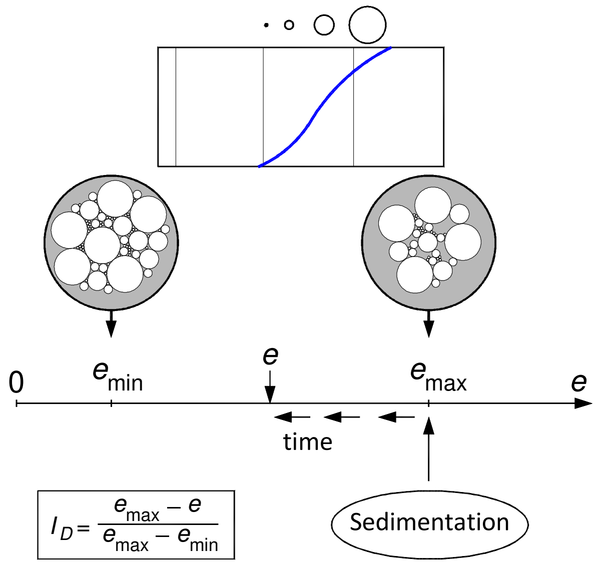


**Figure 1. Grain size distribution *vs.* void ratio interval amplitude (adapted from Matos Fernandes, 2017)**

As to what concerns item i), one should bear in mind that the test for the determination of $e_{max}$ is a laboratory simulation, naturally simplified, of the sedimentation process. This fundamental aspect is seldom emphasized in most textbooks. In complement, the test for determining $e_{min}$ intends to replicate an intense and repeated natural loading process, by dynamically combining vibration and compression.

After discussing the questions previously listed, it is natural and appropriate to highlight, for the first time in the course, the importance of the soil stress history in its mechanical behaviour and, then, to conclude that ancient soils typically tend to be more sound than recent soils. With a small additional step, the site geological scenario can be associated, by adding that Holocene age sand deposits mostly comprise soils with low density indices. And, depending on the geographic conditions, to comment on what happens in successively more ancient formations, from the Plio-Pleistocene age, the Miocene age, etc. In complement, it is simple and timely to explain how recent deposits exhibit deficient behaviour under seismic loading (mentioning settlements and leaving liquefaction for a later occasion, for obvious reasons) and to refer, for the first time, to the methods of treatment that may prevent such behaviour, while also improving the response to static loading.

## 4 Clayey soils

Figure 2 schematically illustrates the Atterberg limits, controlled by the fine fraction and its mineralogical type. In the authors' opinion, in conjugation with the introduction of the Atterberg limits, the following essential points should be immediately added for a preliminary understanding of the mechanical behaviour of sedimentary fine plastic soils:

i) at the "moment" of sedimentation, each soil approximately assumes its liquid limit, $w_L$;

ii) as a result of natural loading conditions (the weight of new sediments), the void ratio progressively decreases;

iii) the void ratio decrease implies the reduction of the water content, which progressively deviates from $w_L$;

iv) this structural alteration is essentially preserved, even when Nature removes by erosion the overlying layers whose weight led to such evolution;

v) the reduction in water content, as expressed by the increase of the consistency index, $I_C$, has an immediate mechanical effect: it increases the stiffness (and strength) of the soil.

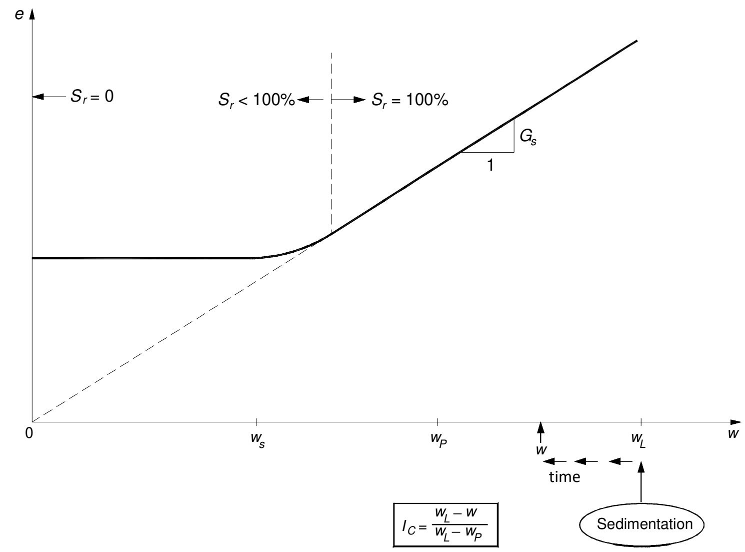


**Figure 2. Atterberg limits (adapted from Matos Fernandes, 2017)**

In relation to item i), one should discuss why the tests for the determination of $w_L$ (Casagrande and fall-cone tests) do not involve a laboratory simulation of soil sedimentation – as opposed to $e_{max}$ for sands. The reason is the practical infeasibility of such simulation for very fine soils. The (rather peculiar!) above mentioned tests have been conceived for a fast and simple identification of the water content for which the soil consistency is extremely low. Therefore, rather than stating that $w_L$ is the water content after sedimentation, it is more appropriate to say that immediately after sedimentation each soil approximately assumes a water content value close to its liquid limit. It is well known that important exceptions exist to this statement, with quick clays being the more notorious. Such exceptions, which may be treated at a later stage in the course, should not prevent this association to be emphasized and their logical consequences to be extracted.

In a similar manner to what has been discussed for sands, it is opportune to identify the importance of the soil stress history on the mechanical behaviour, by outlining that ancient soils tend to be typically more firm than recent soils. This is just a small step away from associating the site geological scenario, by adding that Holocene age clay deposits mostly comprise soils with low consistency indices. And, depending on geographic conditions, to comment on what happens in formations progressively more ancient, from the Plio-Pleistocene age, the Miocene age, etc.

Observing and commenting upon subsoil profiles, namely showing the evolution in depth of the water content and its position in relation to the $w_L$-$w_P$ interval, such as represented in Figure 3, may be very useful in this context (Lambe and Whitman, 1979; Burland, 1990). The same can be said about Figure 4, which collects the sedimentation-compression curves (Terzaghi, 1941) of 20 normally consolidated deposits, from extremely recent muds to late Pleistocene age soils over 1000 m deep, and highlights the consolidation of clay by gravitational loading (Skempton, 1969).

At this early phase, it is not difficult to explain that, such as in Nature the process of reduction of water content/void ratio is very slow, the same happens when a very recent clayey layer, thus located close to the surface, is loaded by a Civil Engineering structure. And to make a first reference to methods that permit to accelerate this volumetric deformation, after explaining that in most cases time-delayed settlements compromise the normal exploration of works.

## 5 Residual soils

Taking into account the regional importance of residual soils from granite in NW Portugal, this preliminary stage of the course also presents a discussion about their typical physical indices, as well as their specificities when compared with sedimentary soils (extreme heterogeneity, cemented structure, influence of relict joints) and their behaviour trends (Viana da Fonseca et al., 1994).

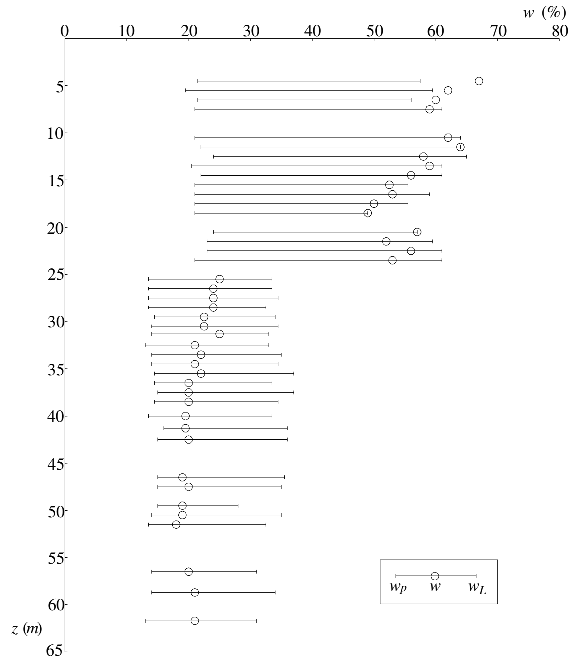


**Figure 3. Water content and liquid and plasticity limits over depth, Troll Oil Field, North Sea, Norway Coast (adapted from Burland, 1990)**

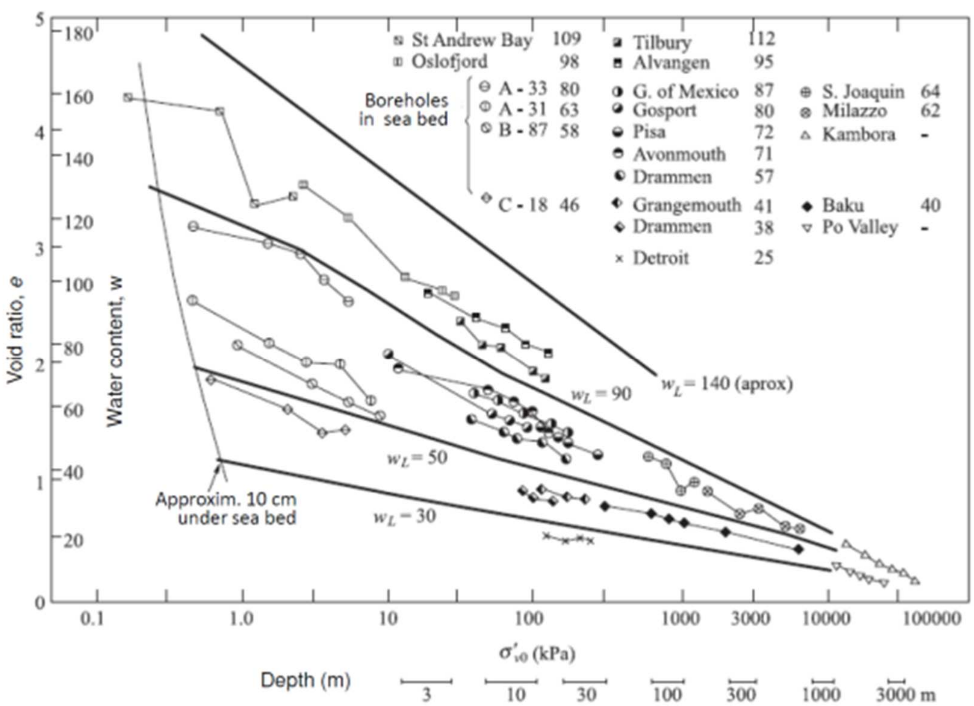


**Figure 4. Sedimentation compression curves from normally consolidated fine sediments (Skempton, 1969)**

## 6 Problem sheets: Examples

Annex 1 includes two examples of problems presented to the students about the soil physical indices and the behaviour patterns previously discussed. These problems are proposed for the 2nd and 3rd weeks of a semester course of 13 weeks.

This form of association of physical indices and geological context with trends concerning mechanical (and also hydraulic) behaviour is developed and extended as this behaviour is treated in subsequent chapters with a truly mechanical approach.

This may be ascertained by the example included in Annex 2, extracted from a final exam. It can be seen that the questions involve aspects such as: i) permeability; ii) normally consolidated and overconsolidated soils; iii) (positive or negative) dilatancy; iv) liquefaction potential; v) evolution with depth of undrained shear strength; vi) solution techniques to accelerate consolidation or increase the density of loose granular soils; vii) foundation soil failure under undrained loading. A proposal for answering those questions is included at the end of the annex.

## 7 Conclusions

In the paper, a gap has been identified in the traditional process of teaching/learning Soil Mechanics.

This gap limits the understanding that the mechanical behaviour – expressed by a series of abstract concepts – is totally controlled by the physical/geological soil characteristics and these physical/geological characteristics are much easier to realise because they are intrinsically concrete!

Most of the main decisions of an experienced engineer are made on the basis of the interpretation of the site geology and of the physical/identification parameters of the relevant soil layers.

The characterisation via mechanical lab and field tests and the calculations are essential in design, but seldom lead to significant changes in the conception of the solution based on the aforementioned interpretation.

The acquisition of expertise to assess the “field atmosphere” usually requires years of experience but can be prepared at the University. This requires training for the ability to interpret the geological conditions and the physical-identification indices and to associate them to trends of the soil mechanical behaviour. This training should begin even before studying the approaches that quantitatively characterise the mechanical soil behaviour. But it should continue and be improved in parallel with these approaches!

This strategy has a number of relevant advantages:

- it trains the *eagle eye*: much can be extracted from the physical indices to assess the expected mechanical trends;
- it establishes an impressive background for the subsequent (mechanical) chapters, whose subjects become more “realistic”;
- it is a good opportunity to introduce solutions to prevent undesirable soil behaviour (just the basic idea);
- it gives rise to very vivid classes, in which students gain enthusiasm because they discuss real engineering problems;
- those simple but powerful ideas are easier to remain retained in the future, as a general knowledge.

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## Annex A

**Example 1** - Figure A1 displays a formation of sedimentary origin over which a petrochemical complex will be constructed. The top layer corresponds to an existing fill placed about 50 years ago.

The project will include a new fill, of very large dimensions in plan, which will raise the soil surface from elevation +2.00 to elevation +4.00. Over this extended embankment, oil storage tanks will be constructed. Such structures are tolerant to moderate foundation settlement. The site is within a seismic zone.

Table A1 shows the physical and identification characteristics obtained from samples taken from the three layers underlying the ancient fill. The order of the soils in the table and the succession of the layers in the figure are not necessarily coincident. Take $\gamma_w$ = 9.8 kN/m$^3$. Assume that all soils are saturated.

**Table A1. Soil parameters**

| Soil | $e_{min}$ | $e_{max}$ | $w_L$ (%) | $w_P$(%) | $\gamma_s$ (kN/m$^3$) | $w$ (%) |
|---|---|---|---|---|---|---|
| 1 | - | - | 50 | 25 | 26.0 | 22 |
| 2 | - | - | 70 | 30 | 25.7 | 65 |
| 3 | 0.28 | 0.90 | - | - | 26.1 | 15 |

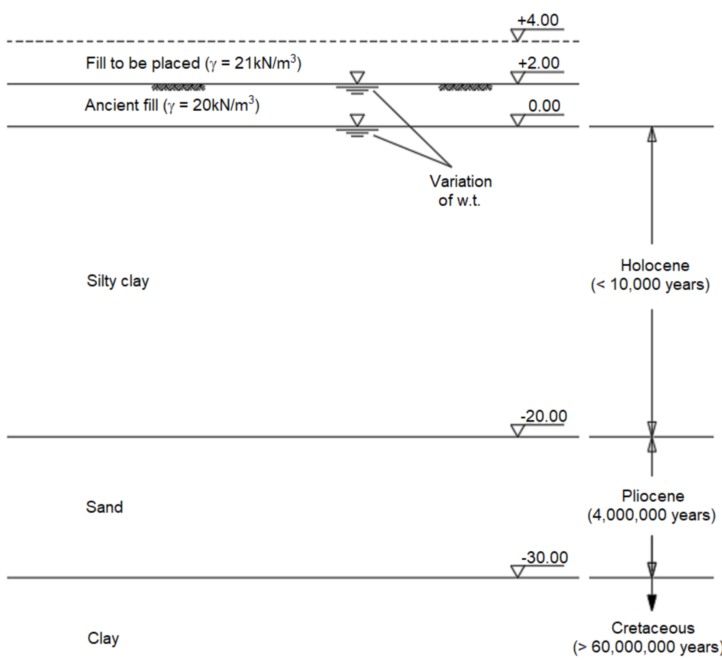


**Figure A1. Geological-geotechnical profile**

a) Calculate the void ratio and the unit weight of the three soils of Table A1. Present the deduction of the expressions employed.

b) Establish the correspondence that you find more reasonable between the layers of Figure A1 and the soils of Table A1 and describe them using at most one line of text for each soil. Present all the parameters required for your answer and the respective calculation. Justify.

c) Select one of the parameters of Table A1 and describe how can be carried out its experimental determination.

d) In case of occurrence of a strong earthquake, will any of the soils exhibit deficient behaviour? In the affirmative case, identify the soil(s) in question and justify. Describe that behaviour and explain how it can be prevented.

e) Due to the placement of the embankment, will any of the soils have large and delayed settlement? In the affirmative case, identify the soil(s) in question and justify. Describe a procedure for preventing such behaviour.

f) Which of the soils of the table would you select as adequate fill material for the construction of the embankment? Justify.

**Example 2** - Figure A2 represents the geological-geotechnical profile of a site where a 30 m high earth fill dam will be constructed. The bedrock consists of granite whose upper zone is weathered. The contact zone of the granite rock with the overlying soil layer C is very irregular, which suggests that this layer might be a residual soil.

Table A2 presents some physical characteristics of the constituent soils of the three layers. Figure A3 displays grain size distribution of the soils of the table. Note that the order of the soils in Table A2 and in Figures A2 and A3 does not necessarily coincide. Assume that all soils are saturated. Take $\gamma_w$ = 9.8 kN/m$^3$.

**Table A2. Soil parameters**

| Soil | $w_L$ (%) | $w_P$ (%) | $\gamma_s$ (kN/m$^3$) | $e_{min}$ | $e_{max}$ | $w$ (%) |
|---|---|---|---|---|---|---|
| 1 | --- | --- | 26.1 | 0.40 | 0.98 | 19 |
| 2 | 34 | 25 | 25.8 | --- | --- | 23 |
| 3 | --- | --- | 26.0 | 0.20 | 0.89 | 18 |

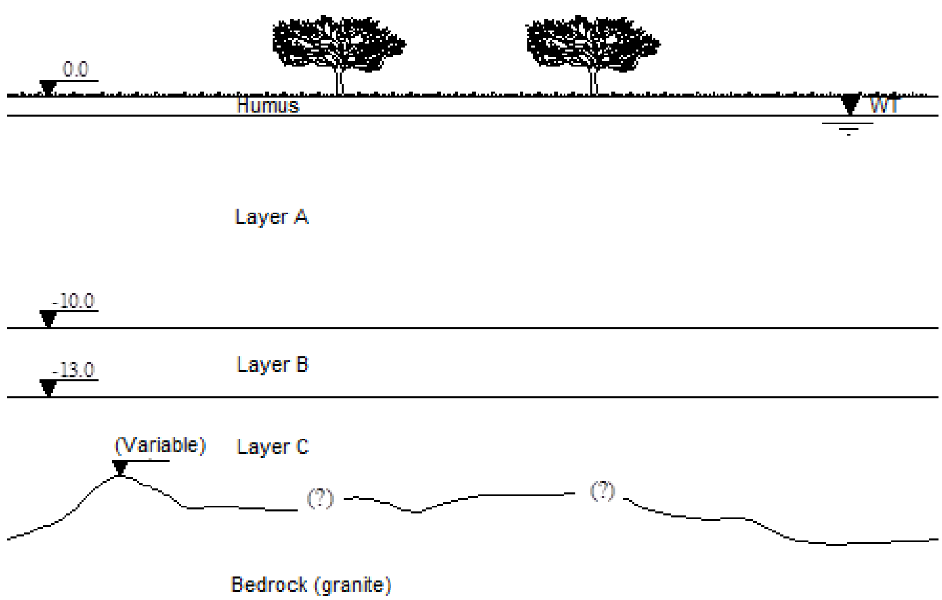



**Figure A2. Geological-geotechnical profile**

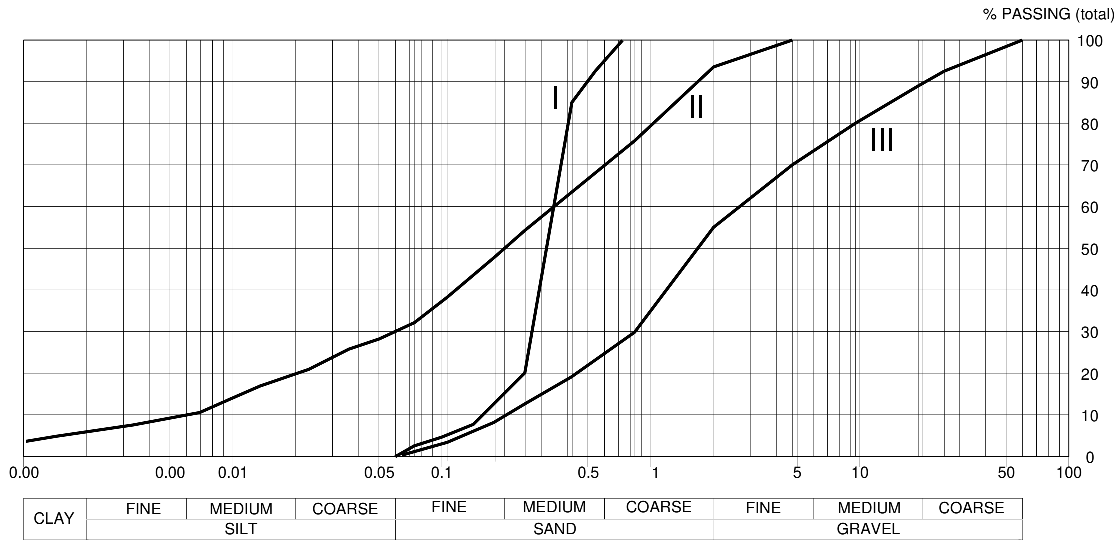



**Figure A3. Grain size distribution curves**

a) Establish the correspondence between the soils 1 to 3 of Table A2, the layers A to C of Figure A2 and the grain size distribution curves I to III of Figure A3. Present the deduction of the expressions employed. Justify.

b) Describe each of the soils for Civil Engineering purposes, using at most six words.

c) In case of occurrence of a strong earthquake, will any of the soils exhibit deficient behaviour? In the affirmative case, identify the soil(s) in question and justify. Describe that behaviour and explain how it can be prevented.

d) Will any layer exhibit large and delayed settlements due to the load applied by the dam? In the affirmative case, identify the soil(s) in question and justify. Describe a procedure for preventing such behaviour.

## Annex B

**Example 3** - Figure B1 presents the geological-geotechnical profile of a geologically very recent alluvial valley that is going to be crossed by a railway line. Part of the line will be constructed over an embankment and part on a bridge with pile foundation. The work is located within a seismic zone. Figure B2 shows the soil layout in the embankment zone.

Table B1 provides physical parameters determined from samples collected in the four soil layers. The order of the soils in Table B1 and in Figures B1 and B2 does not necessarily coincide. Note that in Figure B1 the horizontal scale is much smaller than the vertical scale.

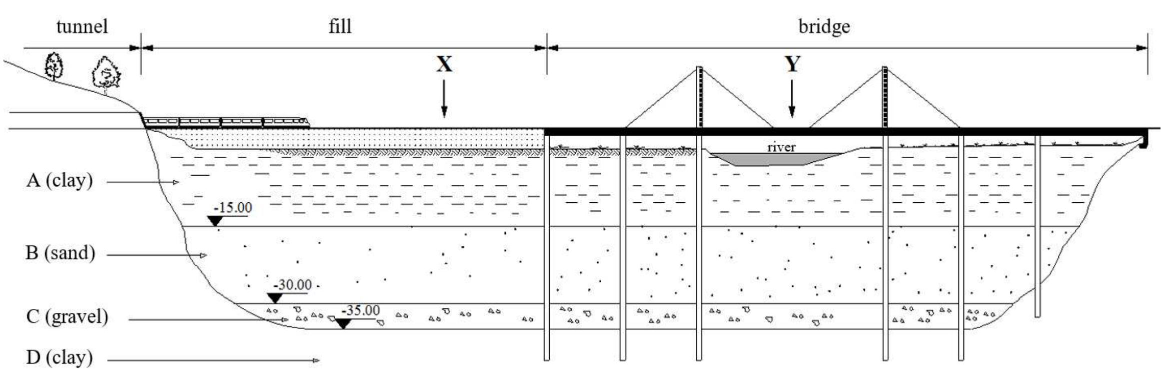



**Figure B1. Geological-geotechnical profile**

**Table B1. Soil parameters**

| Soil | % clay | % silt | % sand | % gravel | $\gamma_s$ (kN/m$^3$) | $\gamma$ (kN/m$^3$) | $e_{min}$ | $e_{max}$ | $w_L$ (%) | $w_P$ (%) |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 0 | 5 | 83 | 12 | 26.0 | 18.5 | 0.25 | 0.95 | - | - |
| 2 | 0 | 0 | 4 | 96 | 25.8 | 20.5 | 0.36 | 0.89 | - | - |
| 3 | 40 | 45 | 15 | 0 | 26.3 | 15.0 | - | - | 88 | 40 |
| 4 | 55 | 35 | 10 | 0 | 26.1 | 20.9 | - | - | 53 | 22 |

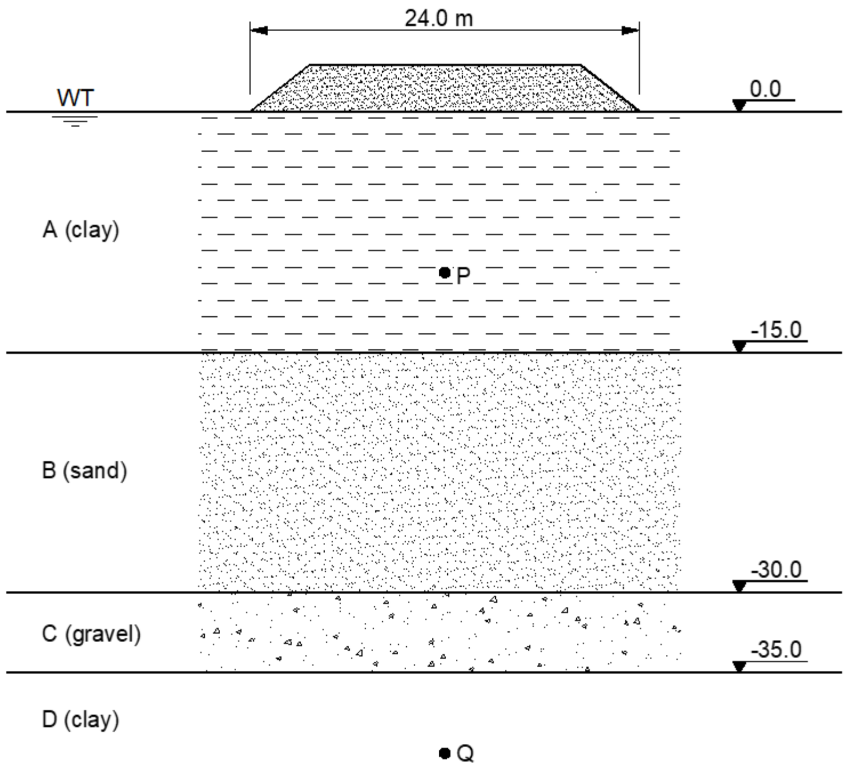



**Figure B2. Soil layout in the embankment zone**

a) Calculate the void ratio and the water content of the soils of Table B1. Present the derivation of the expressions employed. Admit that all soils are saturated. Take $\gamma_w$ = 9.8 kN/m$^3$.

b) Establish the correspondence that you find more reasonable between the soils of Table B1 and the layers of Figures B1 and B2. Present the calculation of the parameters utilised to establish the correspondence.

c) Are the clay fractions of the soils 3 and 4 of the same mineralogical type? Justify.

d) Sort the four soils in increasing order of permeability. Justify.

e) Will any of the soils be probably heavily overconsolidated? How could you ascertain experimentally in the lab your answer? How would have to be the experimental result in order to confirm the overconsolidation?

f) Will any layer exhibit large and delayed settlements due to the construction of the embankment? In the affirmative case, identify the soil(s) in question and justify. Describe a procedure for preventing such behaviour.

g) In case of occurrence of a strong earthquake, may any of the soils exhibit deficient behaviour? In the affirmative case, identify the soil(s) in question and justify. Describe that behaviour and explain how it can be prevented.

h) Sketch the vertical evolution with depth of the undrained shear strength, $c_u$, of layer A along line X (in the tidal flat) and along line Y (in the riverbed) of Figure B1 before the placement of the fill. Indicate a plausible interval for the value of the undrained shear strength at point P of Figure B2, at 10 m depth.

i) Consider point Q located in layer D. Will the value of $c_u$ at point Q be close to, lower than or larger than the value that would be obtained by extending the line drawn in the previous question to layer D? Justify.

j) Classify the four soils as to what concerns the expected dilatancy (positive or negative). How could you experimentally confirm your reply in the lab?

k) When will the safety relative to a rotational embankment and foundation soil failure be minimum: immediately after embankment construction or in the long term? Justify.

### Solution Guidelines for Example 3

a) Based on the values of $\gamma$ and $\gamma_s$, the void ratio, *e*, and the water content, *w*, can be obtained from $G_s w = S_r e$ and $\gamma = \gamma_s (1+w)/(1+e)$, where $G_s = \gamma_s/\gamma_w$ and $S_r = 100\%$ for all soil layers. The results are presented in columns 8 and 9 of Table B2.

**Table B2. Results**

| Soil | $\gamma_s$ (kN/m$^3$) | $\gamma$ (kN/m$^3$) | $e_{min}$ | $e_{max}$ | $w_L$ (%) | $w_P$ (%) | $e$ | $w$ (%) | $I_D$ (%) | $I_C$ | $A_t$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 26.0 | 18.5 | 0.25 | 0.95 | - | - | 0.86 | 33 | 13 | - | - |
| 2 | 25.8 | 20.5 | 0.36 | 0.89 | - | - | 0.50 | 19 | 74 | - | - |
| 3 | 26.3 | 15.0 | - | - | 88 | 40 | 2.18 | 81 | - | 0.15 | 1.20 |
| 4 | 26.1 | 20.9 | - | - | 53 | 22 | 0.47 | 18 | - | 1.13 | 0.56 |

b) Column 10 of Table B2 displays the values of the density index, $I_D$, for the granular soils 1 and 2, while column 11 presents those of the consistency index, $I_C$, for the clayey soils 3 and 4.

Soil 1 is very loose and soil 2 is dense. Soil 3 is very soft, while soil 4 is very stiff/hard.

Taking into account that the density and the consistency increase with the age of the deposit, it may be concluded that the more reasonable correspondence between the layers of Figures B1 and B2 and the soils of Tables B1 and B2 is:

- Layer A: soil 3, very soft silty clay;
- Layer B: soil 1, very loose sand;
- Layer C: soil 2, dense gravel;
- Layer D: soil 4, very stiff/hard clay.

c) Column 12 of Table B2 presents the values of the activity of clay, $A_t = I_P/(\%\ clay)$, which show that the clay fractions are not of the same type, with that of soil 3 being more active.

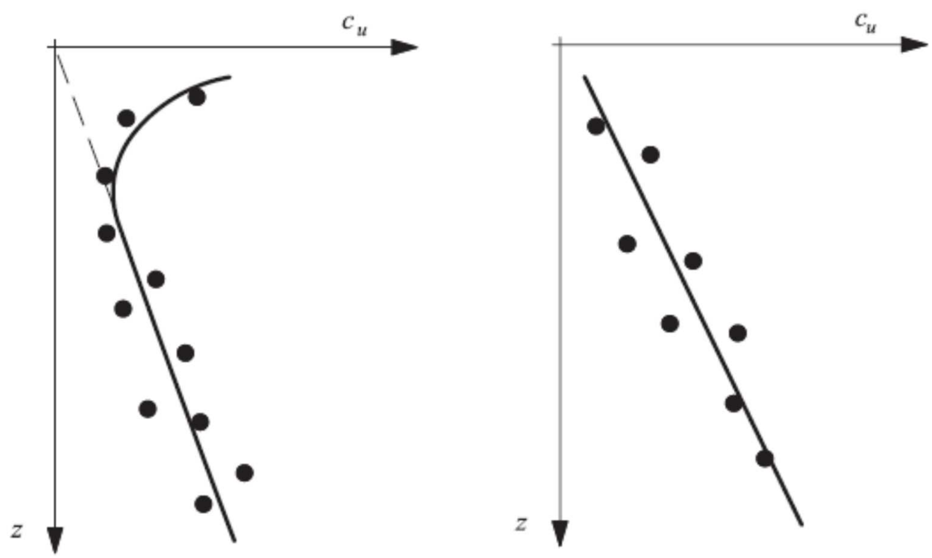
d) The finer the soil, the lower the permeability. So: $k_4 < k_3 < k_1 < k_2$.

e) Clay layer D, given its deep location in Figure B1 and its high consistency, may be highly overconsolidated. This prediction could be checked by performing oedometer tests on undisturbed samples. These would allow to estimate the maximum past vertical effective stress experienced by the soil. In case it significantly exceeds the at rest effective vertical stress, the prediction is confirmed.

f) Layer A, a very soft clay 15.0 m thick, may probably experience large and delayed settlement by consolidation. The consolidation rate can be significantly increased by means of a grid of vertical drains that reach sand layer B.

g) As B is a layer of very loose sand under the water table, two problems may occur: i) large settlement due to the vibration induced reduction of void ratio; ii) liquefaction, which may cause even more serious damage due to the dramatic reduction of soil strength. It would be appropriate to increase the density of the layer by vibro compaction.

h) Since layer A is a soft clay in this geologically very recent alluvial valley, it is very likely normally consolidated, with the undrained shear strength proportional to the at rest effective vertical stress, increasing linearly with depth. The difference between the (permanently submerged) riverbed and the tidal flat is that in the latter, due to the emersion-submersion cycles associated with the seasonal variations of the water table, a surface crust develops by desiccation whose undrained strength is higher. Figure B3 presents the evolution with depth of $c_u$ in the two zones. The $c_u/\sigma'_{v0}$ ratio lies typically within the [0.20, 0.40] interval. This interval is, in part, a consequence of the anisotropy of the undrained shear strength. Assuming the water level coincident with the ground surface, at point P, $\sigma'_{v0}$ = 52 kPa. Therefore, a plausible interval for $c_u$ is [10 kPa, 20 kPa].




(a) (b)

**Figure B3. Evolution of $c_u$ with depth: a) tidal flat (section X); b) riverbed (section Y)**

i) Since layer D is probably overconsolidated, the undrained shear strength at point Q will be larger, or likely much larger, than the value obtained by simply extending the line drawn for layer A.

j) The soft clay and the loose sand will probably exhibit negative dilatancy (volume reduction), while the dense gravel and the very stiff/hard clay will probably experience positive dilatancy (volume increase). This could be confirmed by performing triaxial tests on undisturbed samples.

k) During the consolidation process subsequent to loading, the (positive) excess pore pressure dissipates, the average effective stress increases and the shearing stress remains practically constant. Therefore, the shear strength increases at each point of the clay layer A until the end of consolidation. This is why stability analyses must be carried out for the conditions prevailing at the end of construction, assuming undrained conditions.

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