Difficult foundation conditions in Romania

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ABSTRACT

Significant areas of the Romanian territory are covered by soils that impose special foundations. From the category of difficult soils the following have a particular behaviour in relation to water: loessoid soils and swelling -shrinkage soils. Loessoid soils collapse irreversibly when are saturated, leading to significant settlements, while swelling - shrinkage soils significantly change their volume when moisture variations occur. In both cases, measures should be taken either to avoid water infiltration or for desensitization of these soils in relation to water. The paper presents significant aspects from the technical norms related to elaboration of geotechnical documentations (NP 074-2014) and foundation of constructions on loessoid soils (NP 125-2010) and swelleing - shrinkage soils (NP 126-2010). The paper also presents the results of several tests performed on: expansive clay mixed with sand, gravel and slag (foundry waste), loess mixed with sand, bentonite mixed with sand (bentonite enhanced sand) and loess mixed with bentonite enhanced sand.

Keywords: collapsible soils, loess, expansive soils, soil stabilization

1. INTRODUCTION

Significant areas of the Romanian territory are covered by soils that impose special foundation solutions. These solutions may consist in adopting a deep foundation system (pilots, barrettes), improving the natural foundation soil, taking special measures regarding water seepage in the foundation soil, taking measures to ensure the stability of the site, etc ...

For these sites, the adequate study of the physical and mechanical properties of the foundation soil represents a key issue in the geotechnical design.

2. GEOTECHNICAL DOCUMENTATIONS FOR CONSTRUCTIONS

In Romania, the geotechnical research of the foundation soil performed in order to elaborate geotechnical documentations for-constructions is done according to NP 074-2014 "Technical norm on geotechnical documentations for constructions ".

In order to establish the requirements of field investigations, laboratory testing and geotechnical design, this regulation introduces 3 geotechnical categories for which geotechnical risks are associated.

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Table 1. Geotechnical categorization

Geotechnical risk	Points limits	Geotechnical category
Low	6...9	1
Moderate	10...14	2
Major	15...22	3

The geotechnical risk depends on two categories of factors: on the one hand factors related to the soil, of which the most important are the soil conditions and groundwater, and on the other hand, factors related to the characteristics of the construction and its neighborhoods.

In order to define the geotechnical category / risk are evaluated the following factors:

1. soil conditions:
   - good soils - 2 points
   - average soils - 3 points
   - difficult soils - 6 points

Table 2. Difficult soils according to the Romanian legislation (NP 074-2014)

No.	Type of soil
1	Sandy soils, including silty sands, in loose state
2	Saturated sandy soils, sensitive to liquefaction under seismic loads
3	Fine soils with consistency index $I_C < 0.5$
4	Loessoid soils in group B of sensitive to wetting soils defined by NP 125
5	Swelling – shrinkage soils (active clays) defined by NP 126
6	Soils with high content of organic matter (>6%)
7	Slope sites with potential to landsliding
8	Uncontrolled soil fills, less than 10 year old
9	Waste fills, no matter how old

2. groundwater:
   - no dewatering - 1 point
   - normal dewatering system - 2 points
   - exceptional dewatering system - 4 points
3. classification of buildings by importance:
   - low - 2 points
   - normal - 3 points
   - special, exceptional - 5 points
4. neighborhoods:
   - no risk - 1 point
   - moderate risk - 3 points
   - major risk - 4 points
5. seismic zone:
   - $a_g < 0.15g$ - 1 point
   - $a_g = (0.15...0.25)g$ - 2 points
   - $a_g > 0.25g$ - 3 points

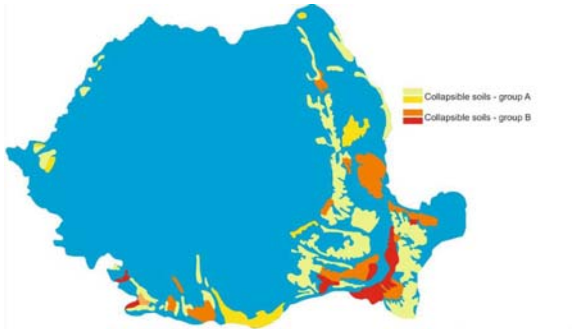


Figure 1. Map of collapsible soils in Romania

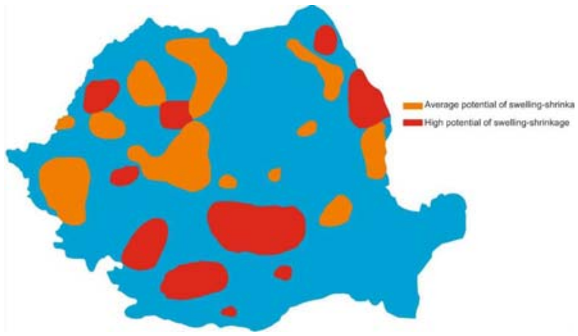


Figure 2. Map of swelling – shrinkage soils in Romania

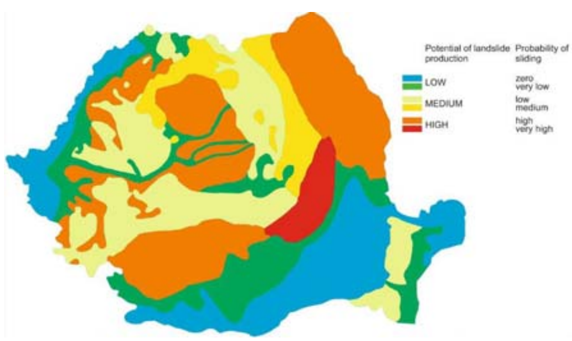


Figure 3. Territory macrozonation concerning landsilde hazard

It can be observed that the existence in a site of a difficult foundation soil (6 points) cumulated with the other 4 factors, will lead to a minimum score of 11 points, which will frame the site in geotechnical category 2 / moderate risk.

Geotechnical categories 2 and 3 impose in situ researches, laboratory tests and design methods more elaborated than those recommended for geotechnical category 1.

In the figures above are presented maps of the Romanian territory with collapsible soils (Figure 1), swelling – shrinkage soils (Figure 2) and potential of landslide production (Figure 3).

3. COLLAPSIBLE SOILS

In Romania the loessoid (collapsible) soils covers about 17% of the territory as it is indicated in the map from Figure 1.

The soils sensitive to wetting are defined as unsaturated macroporic cohesive soils, which in contact with water are subjected to sudden and irreversible changes of internal structure, reflected by additional settlements and decreases of mechanical geotechnical parameters.

Additional settlement may occur under the own weight of wetted layer (I_{mg}) and under the action of compressive loads transmitted by the foundations (I_{mp}).

In terms of how the settlement occurs, the loess is classified in two groups:

- Group A: loess having additional I_{mg} less than 5 cm;

- Group B: loess having additional I_{mg} equal to or greater than 5 cm.

The minimum specific geotechnical data necessary to classify a soil as sensitive to wetting are related to composition and compressibility in natural and saturated conditions.

In this connection were imposed physical (I) and mechanical (II) identification criteria as follows:

I.1. cohesive soil with silt 50 ÷ 80% in unsaturated state (Sr <0.8) and the natural porosity n> 40%.

I.2. Index $I = \dfrac{e_L - e}{1+e}$ with values between 0.10 ÷ 0.30 depending on the plasticity index Ip = 10 ÷ 22%, where e is the void ratio in natural state and e_L is the void ratio at the liquid limit of plasticity, w_L of the soil.

II.1. the index of additional settlement to wetting under the load of 300 kPa (in oedometric test i_{m300} > 2%).

II.2. the indexes η and δ related to soil settlement in natural and flooded state (in plate load test) have values:

$$\eta = \frac{s_i}{s_n} \geq 5 \text{ şi } \delta = s_i - s_n \geq 3 \text{ cm},$$

where s_i is submerged soil settlement and s_n is the settlement at natural moisture content as determined by plate load test under the pressure of 300 kPa.

To characterize a soil as sensitive to wetting should have at least one criteria related to physical properties and one criteria related to mechanical behavior.

It follows that for the soils sensitive to wetting the oedometric tests should be made after a specific methodology, being recommended double tests - on samples with natural moisture content and on saturated samples - and/or samples at natural moisture content, saturated under the pressure of 300 kPa. (Figure 5).

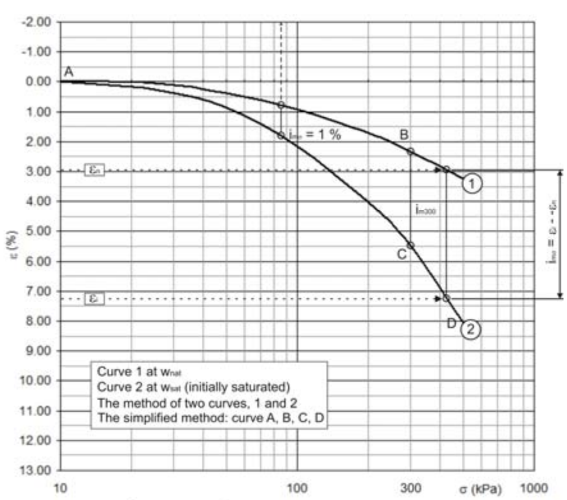


Figure 4. Specific oedometric tests on loess. Double curves method.

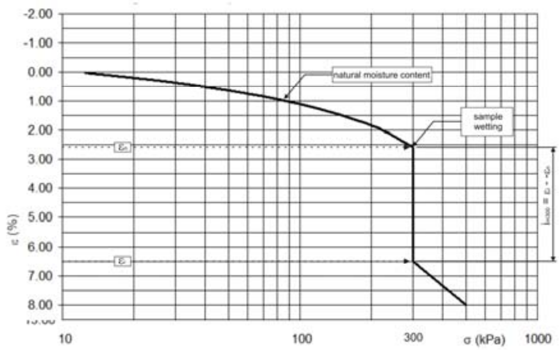


Figure 5. Specific oedometric tests on loess. Single test method

Thus, are obtained the necessary parameters for geotechnical design, namely, the index of additional settlement to wetting $i_{m\sigma}$, including i_{m300}, and the structural resistance σ_0 which represents the pressure at which $i_{m\sigma}$ is equal to 1%.

It is also necessary to determine the shear strength parameters on samples with natural moisture content and saturated. In Table 1 are presented indicative characteristic values for loess and loess-like soils in Romania for the main geotechnical parameters.

Table 3. Typical values for geotechnical parameters of loess and loessoid soils

Geotechnical parameter	Characteristic value
Skeleton density, ρ_s [g/cm^3]	2,52 - 2,67
Unit weight of the soil, γ [kN/m^3]	12,0 - 18,0
Dry unit weight of the soil, γ_d [kN/m^3]	11,0 - 16,0
Porosity, n [%]	40 - 55
Plasticity index, I_P [%]	5 – 22
Index of additional settlement to wetting σ = 300 kPa, i_{m300} [%]	2 – 14
Oedomertic modulus, $E_{oed\ 200\text{-}300}$ [kPa]	5000 - 15000
Internal friction angle, φ [°]	5 – 25
Cohesion, c [kPa]	10 - 30

3.1. Settlement calculation

In the case of loessoid foundation soil it is compulsory to calculate the settlement in the hypothesis of foundation soil wetting.

Additional settlement may occur under the own weight of wetted layer (I_{mg}) and under the action of compressive loads transmitted by the foundations (I_{mp}).

Settlement calculation is performed on entire thickness of the layer sensitive to wetting by dividing it into elementary layers.

For an elementary layer "I" it is evaluated the vertical stress under own weight at the natural state (σ_{gn}) and, from the stress – strain oedometric curve on natural soil sample, the specific settlement (ε_{gn}). The vertical stress under own weight of saturated layer (σ_{gi}) is evaluated and, from the stress – strain oedometric curve of the saturated sample, the specific settlement (ε_{gi}) is determined. The difference between the two settlements represents the specific settlement of the elementary layer "i" under its own saturated weight (i_{mg}) (Figure 6).

The vertical stress under the vertical load transmitted by the foundations (σ_z) cumulated with the stress under own weight in saturated conditions (σ_{gi}) represents the total stress that will act on the elementary layer "I". The difference between the specific settlement of the saturated sample (ε_{pi}) under this effort and the specific settlement under own saturated weight (ε_{gi}) represents the settlement of the saturated soil under the vertical load transmitted by the foundations (i_{mp}) (Figure 7).

If I_{mg} is the settlement under own weight of the saturated layer, I_{mp}+I_{mp} represents the total settlement under own weight of saturated layer and the load transmitted by the foundation. This calculation imposed by the technical norm NP 125-2010 doesn't indicate the settlement of the foundation in the hypothesis the foundation soil is not wetted (s), which is presented in Figure 8.

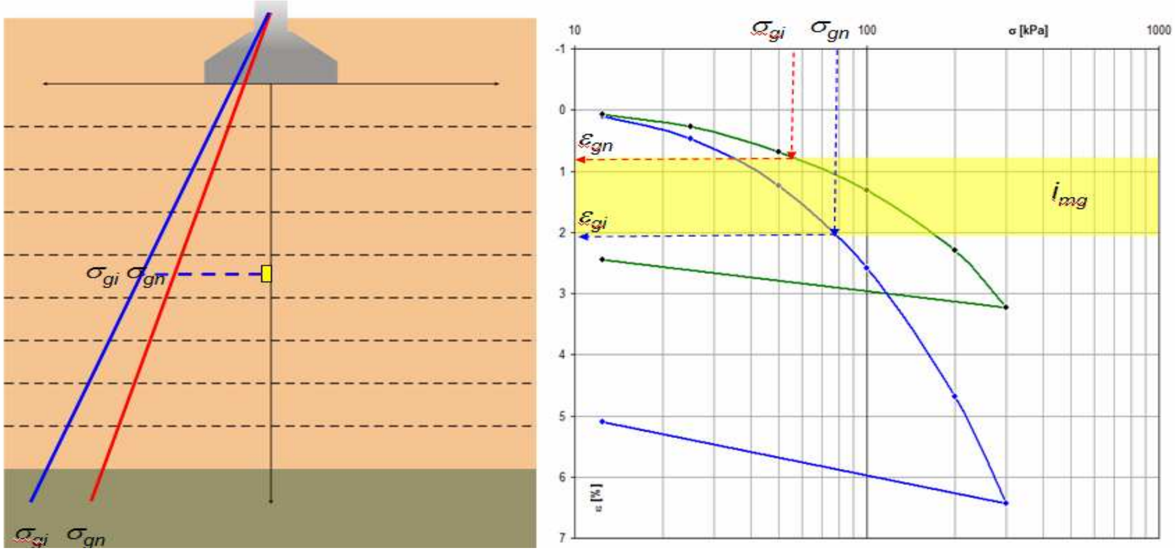


Figure 6. Additional settlement to wetting due to own weight of wetted layer (I_{mg})

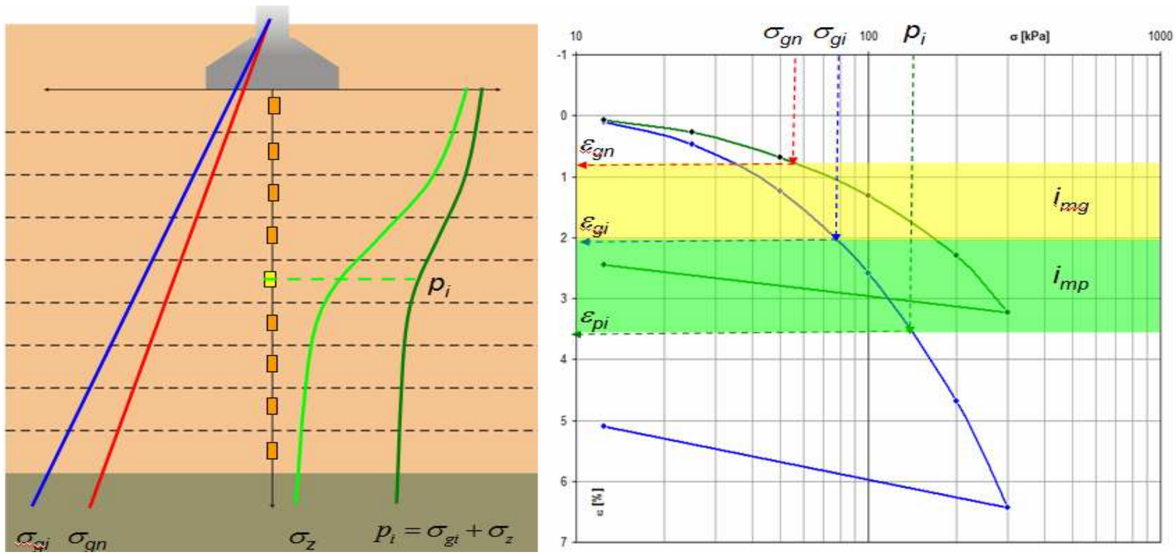


Figure 7. Additional settlement to wetting under external loads (I_{mp})

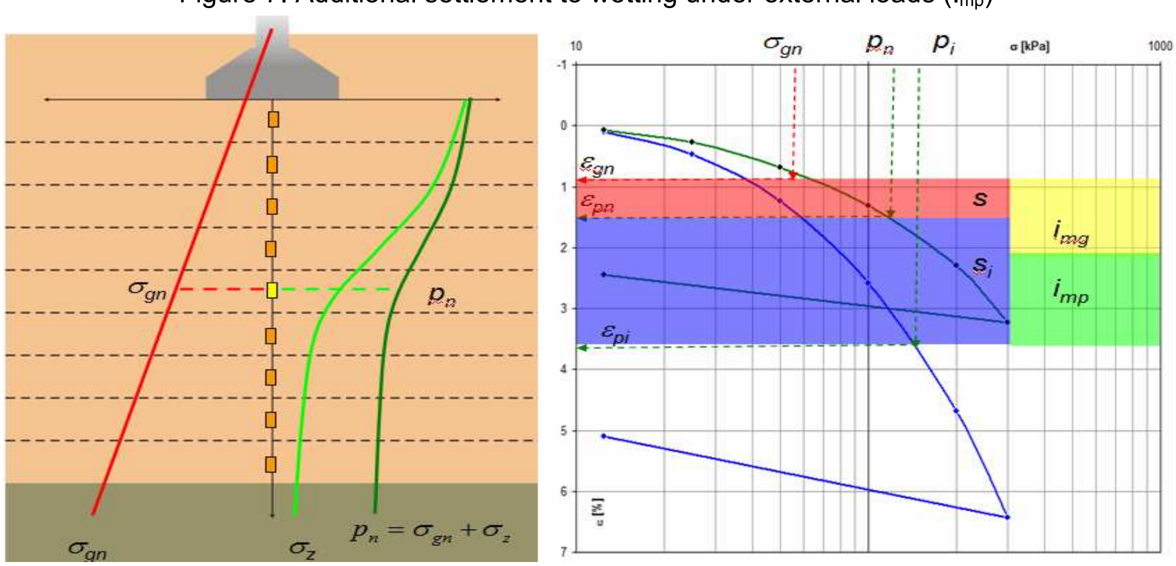


Figure 8. Settlement in natural conditions (without wetting) (s) and settlement after wetting of the foundation soil (s_i)

3.2. Geotechnical design by calculation

For the design of the foundation solutions on collapsible soils, the following will be taken into account:

- for the verification at normal exploitation limit state, differential settlements of the foundations will be limited in order to avoid appearance of any limit state in the structure;
- the compatibility of the deformations reached in ultimate limit state will be taken into account, by analyzing the relative rigidity of the structure and soil;
- the choice of the geotechnical actions, based on the destination and lifetime of the construction, will be considered those resulted from wetting (saturation) of the soil taking into account:

  • the source and the type of the wetting (local, general);

  • the direction of wetting, which can be gravitational or generated by the rising of the groundwater table;

  • speed and direction of the groundwater flow, that can have alternately different directions (irrigational canal, shore);
- in the case of pile foundations embedded in a layer non-sensitive to water, beneath a loess layer, if the wetting is possible and the settlement under the weight of the soil can occur, if will be considered the negative skin friction on the piles.

3.3. Calculation of the foundation soil

By knowing the structural resistance σ_0 it can be defined the zones in the foundation soil where deformations occur.

Therefore, the deformable upper zone extends until the depth where the vertical effort (σ) of the foundation load (σ_z) and the soil weight (σ_{gz}) becomes equal to σ_0 (Figure 9).

On the other hand, for some thicknesses of the collapsible soil layer, additional settlements can occur also at the base of the layer, defined as deformable lower zone, where σ_{gz} is bigger enough to result a total vertical load (σ) bigger than the structural resistance (σ_0). In Figure 9 is indicated a middle zone, named inert zone because the total vertical load is less than the structural resistance, therefore no additional settlements due to soil wetting will occur.

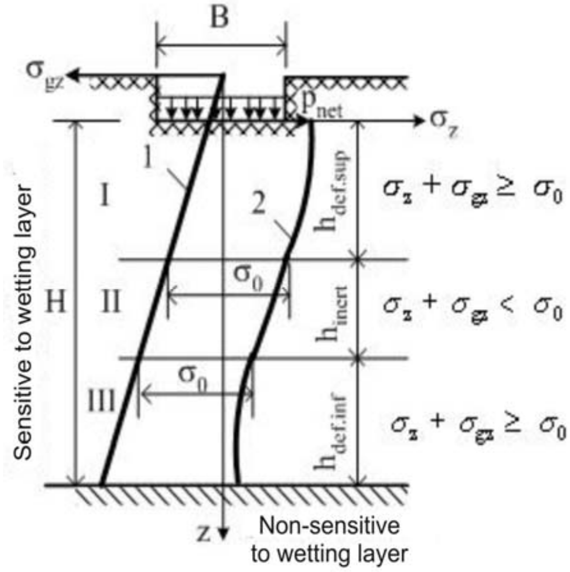


Figure 9. Characteristic zones in the foundation soil composed of collapsible soils

With respect to the foundation width (B) and the value of the net pressure on the foundation raft (p_{net}), the thickness of the layer sensitive to water (H) and the value of the structural resistance (σ_0) other situations can occur (Figure 10 a…e).

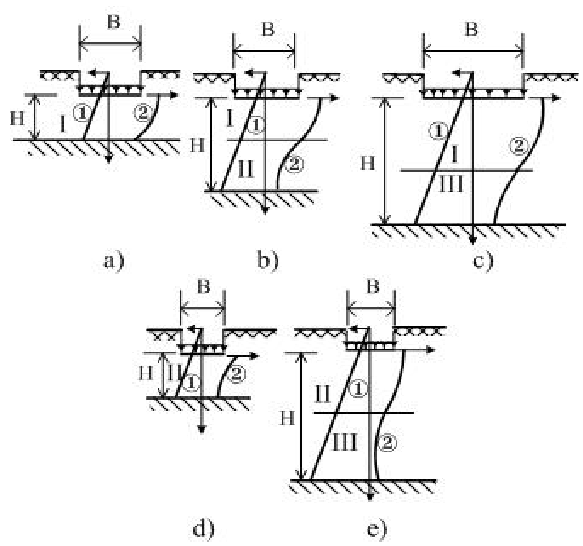


Figure 10. Characteristic situations for the foundation soil composed of a layer sensitive to water

The calculations at the normal exploitation limit state imply verification of the settlements. It will be taken into account: additional settlements to wetting under the weight of the soil (I_{mg}) and under the external loads (I_{mp}) according to the type of the loess (group A or B) and the desensitization measurements.

The calculations at the ultimate limit state refers to the evaluation of the bearing capacity based on the shear strength parameters (φ and c) at natural moisture content and saturated, according to the measurements for the foundation soil.

3.4. Measurements for the choice of the foundation solutions

The following measures should be taken into account for the choice of the foundation solution on a foundation soil consisting of loessoid soils:

- prevention of soil wetting;
- soil improvement by different technologies following the formation of a new internal structure for the entire layer (desensitization to wetting). Can be considered:
  - intensive compaction;
  - injection by silication
  - thermic treatment;
  - compacted columns of concrete or local materials; it is forbidden to use only granular permeable materials.
- construction of a compacted cushion above the layer of collapsible soil; it is forbidden to use only granular permeable materials;
- replacement of the collapsible soil layer by excavation and controlled soil fill with adequate materials;
- consuming of the additional settlements by wetting through:
  - controlled wetting;
  - saturation under supplementary load;
  - deep explosions.
- selection of indirect foundation system (piles, barrets, etc…) embedded in a non-sensitive to water layer.

4. LOESSOID SOILS MIXED WITH SAND AND BENTONITE

In the experimental programme, various mixtures of loessoid material with different natural mineral materials have been proposed, in view of eliminating moisture sensitiveness, improving geotechnical parameters of mechanical behaviour and limiting permeability.

To this purpose, a series of mixtures have been proposed: loess with sand 1-2 mm (Cu = 1.5) and loess with sand and bentonite powder addition in two variants of mixture. The obtained mixtures are presented below:

- Mixture 1: 80% loess + 20% sand (1-2 mm);
- Mixture 2: 60% loess + 40% sand (1-2 mm);
- Mixture 3: 50% loess + 40% sand (1-2 mm) + 10% bentonite;
- Mixture 4: 50% loess + mixture from (40% sand (1-2 mm) + 10% bentonite).

The difference between the last two mixtures consisted in the way they were mixed. In the first case, all the three materials were simultaneously mixed and then water was added to reach different degrees of humidity in order to perform the normal Proctor test. In case of the last mixture, the sand was first mixed with the bentonite and with water and then, after this mixture had dried, it was also mixed with the loess (Burlacu et al. 2013).

As a result of the Proctor test outcome analysis (Figure 6), it has been observed that along with adding up and increasing the percentage of sand in the mixture (from 20% to 40%), the maximum density in dry condition increases. At the same time, the optimal compaction moisture of the mixtures decreases.

In case of mixture 4, the Proctor curve doesn't have a peak but a constant zone for the maximum dry density, which was obtained for moisture content values ranging between 11 and 15%. In order to validate the results, tests on this sample were carried out again and similar values were obtained (Figure 7). The moisture content plays a key role in the real scale of compaction process. Given that, the last indication regarding mixture 4 is important because it allows compaction at wider domain of moisture content.

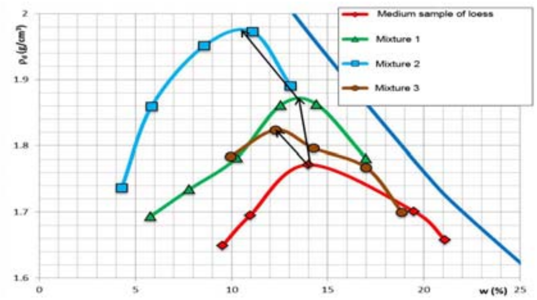


Figure 11. The results of the Proctor trial for all the mixtures obtained

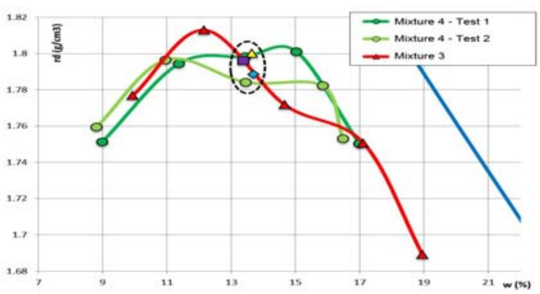


Figure 12. Results of Proctor test for mixtures 3 and 4

As to the values of the permeability coefficient, these have been of the order of 10^{-5} cm/s for the average loess sample rising up to values of 10^{-4} cm/s in case of the mixture containing 40% sand, while in case of the mixtures containing an addition of bentonite, the measured values were below 10^{-9} cm/s.

5. EXPANSIVE CLAYS

Clayey soils have the property to significantly modify their volume when moisture changes: they shrink when moisture reduces and they swell when moisture increases. Due to their shrink-swell behaviour, these soils could create many problems for engineering structures and for this reason, direct foundation is not allowed by the legislation in force, being mandatory to be replaced or improved by stabilization.

Shrink-swell behavior is caused by the mineralogical composition of clay minerals. These minerals determine the natural expansiveness of the soil, and include smectite, vermiculite, illite and chlorite. Generally, the larger the amount of these minerals is present in the soil, the greater the expansive potential. These expansive effects may become “diluted” by the presence of other non-swelling mineral such as quartz and carbonate (Ivasuc et al., 2013).

The activity in relation to water of the expansive soils can be estimated based on physical and mechanical properties which are determined in the laboratory according to the legislation in force. The physical properties that characterize the activity of expansive soils are: content of colloidal clay, plasticity index, activity index and free swell. The mechanical property which indicates more accurately the activity of expansive soils is the swelling pressure, determined in consolidation tests on saturated samples.

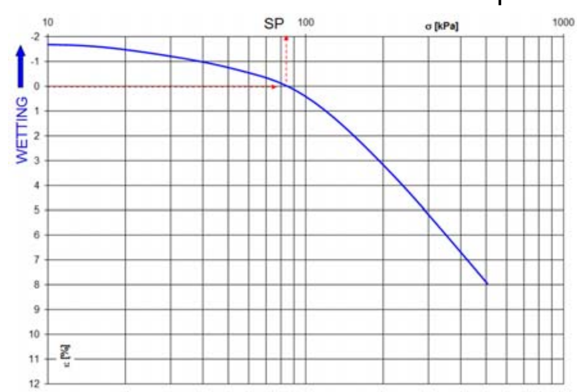


Figure 13. Determination of the swelling pressure. Simple method

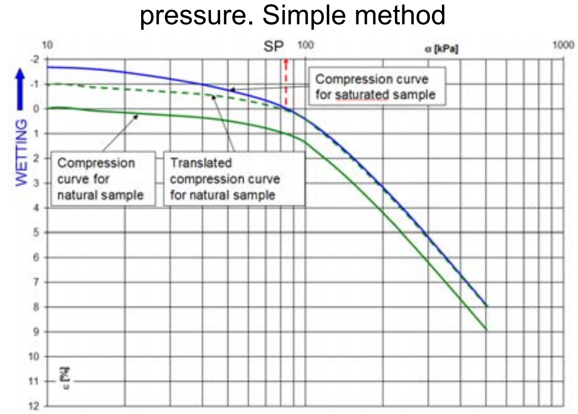


Figure 14. Determination of the swelling pressure. Double method

In Romania, the foundations laid on soils with high shrink-swell potential must comply with the requirements of NP 126:2012. In order to classify a soil in the category of soils with high shrink-swell potential, the following geotechnical parameters are mandatory: $A_{2\mu}$ - the percentage clay content with a diameter less than 0.002 mm (%); I_P - plastic index (%); I_A - activity index; C_P - plasticity criteria (%); U_L - free swelling (%) and SP - swelling pressure (kPa) (NP126-2010).

6. MECHANICAL BEHAVIOR OF DESTRUCTURATED EXPANSIVE CLAY

6.1. Natural soil properties

The studied site is located in the Transylvanian plateau; it is approximately 19.5 ha and the level difference ranged from 265 m to 315 m nMN. In the initial state, the site was not affected by landslides (Olinic et al., 2014).

According to STAS 1913/5-85, the analyzed soils were clayey soils with the grain size distribution composed of 50-70 % clay ($A_{2\mu}$ = 40 – 48 %), 25-40 % silt and 1-10% sand. Determining the plastic and liquid limits, we observed that the plasticity index (I_P) showed values between 26.7 – 57.2 %.

Oedometric compression tests were performed on samples that were initially saturated to determine the swelling pressure (S_P) with recorded values between 40…200 kPa.

The shearing resistance parameters were determined in CU and CD conditions performed on samples with natural humidity and initially saturated samples. Table 4 shows the variation of the shear strength parameters.

Table 4. Shear strength parameters

Parameter/ Shearing test type	Internal friction angle, Φ [o]	Cohesion, c [kPa]
CU	11 ÷ 29	59 ÷ 160
CUsat	19 ÷ 28	23 ÷ 81
CD	19 ÷ 30	33 ÷ 80
CDsat	17 ÷ 23	25 ÷ 55

According to NP 126-2010, the studied soils are classified in the category of very active clays which are considered to be regarded as difficult foundation soils in compliance with NP 074-2014. According to STAS 2914-84, all materials intercepted in the investigated depth were included in category of 'bad' quality soils. In this case, they cannot be used (in their natural state) as filling materials for the body of perimetral embankments for the municipal solid waste landfill or for other fillings.

If these materials are used as filling materials, they must be desensitized in relation to water, to undertake excavations and reshape the slopes to 1:3 and to construct berms with a width of 4...5 m for every 10...12 m on vertical (Ivasuc, 2013).

6.2. Clay destructuration

Clayey soils are very sensitive to environmental conditions, especially to variation in humidity and temperature. Clay destructuration occurs after a series of cycles of hydration-dehydration and freezing-thawing; this modification produces irreversible effects on the structure and texture of the expansive soils causing important damages such as cracks, differential settlements and loss of stability (one of the major problems in earthwork aplications).

All these damages are highly influenced by the water content and degree of saturation: during dehydration the frequency of macropores increases, which during hydration the macropores do not close-up perfectly and hence cause the soil to bulk-out slightly, and also allow enhanced access to water for the swelling process.

Engineering practice has shown that water may enter into every soil structure: from precipitation (rainfall and snow) or from the ground. In order to avoid the exposure to humidity and temperature variations (by preventing the access of water to the embankments) it is recommended to cover the slopes with a layer of top soil and to vegetize it with grass and shrubs (Olinic et al., 2014).

Instability phenomena have appeared on site during the winter of 2013, after the excavation works which have been performed in autumn and the slopes remained unprotected (Figure 15), (Olinic et al., 2014).

November 2012

February 2013

November 2012

February 2013

Figure 15. Loss of stability on the site

6.3. Laboratory tests on destructurated clay

In order to explain the instability phenomena occurring on the site, a representative sample of clay (glomerular clay) was chosen for submission to freezing-thawing cycles (at natural humidity and temperature variations) from November 2012 until February 2013 (Olinic et al., 2014). The phases of clay sample destructuration can be seen in Figure 16.

To verify the effect of destructuration of a clay sample and to explain the loss of the stability appeared on the site (Figure 15), the shear strength parameters ϕ (internal friction angle) and c (cohesion) have been determined by the direct shearing tests which simulated the landsliding phenomena.

08.11.2012 27.12.2012

25.01.2013 02.02.2012

Figure 16. Clay destructuration at different periods of time

The landsliding phenomena correspond to an unconsolidated–undrained (UU) direct shear strength test performed on a saturated sample. For this reason, the test consisted in the application of an 18 kPa contact load, the saturation of the sample for approximately 2h and application of a pressure of 67, 121, 175, 230 and 285 kPa. Direct shearing was performed with the speed of 1 mm/min, the test stimulating the real conditions from the site (Olinic et al., 2014).The shearing strength parameters resulting from the destructured clay analysis are: internal friction angle (Φ = 17.48°) and cohesion (c = 6.94 kPa).

The values of the shear strenght parameters, especially the cohesion, confirm the fact that after destructuration the internal soil structure is affected by the weakening of cohesion between clay particles.

7. EXPANSIVE CLAY STABILIZATION BY MIXING WITH GRANULAR MATERIAL

In order to realize the perimetral embankments from the municipal solid waste landfill, placed on a slope area and built from expansive soils, it was attempted to stabilize the existing clayey material from the site by adding different granular materials.

It was proposed an experimental program consisting of the determination of the following geotechnical properties: optimal compaction parameters, swelling pressure, compressibility and consolidation parameters and shearing resistance parameters in consolidated-undrained conditions on saturated samples (CUsat) around the optimal compaction parameters (Ivasuc, 2013).

In order to stabilize the expansive clay by mixing with a non-cohesive material, it was proposed three types of granular materials: S - slag (foundry sand), SG – sand with gravel and G - gravel with particles of 4-8 mm in diameter.

Based on the compaction tests (normal Proctor test) performed for the natural clay samples and for the mixtures with granular materials, the optimal parameters of compaction showed a decrease in the optimal moisture content and an increase in the dry density with the increasing the percentage of granular material.

Usually, natural clay has swelling pressures that differ significantly from their stabilized samples. In this case, for some addition percentages of non-cohesive materials, the effect of compaction is higher that the desensitization effect, resulting materials considered even ‘worse’ than the natural sample - materials with swelling pressure more higher than swelling pressure of the natural sample (Figure 17).

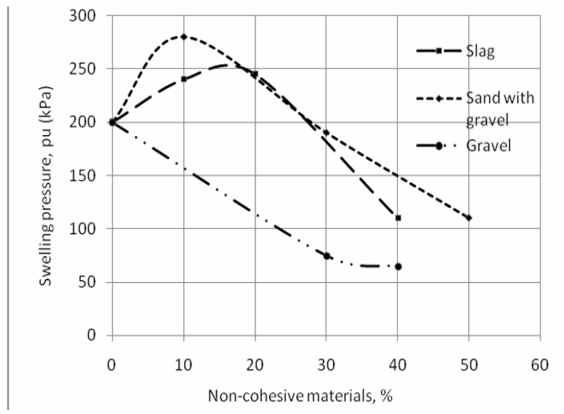


Figure 17. Swelling pressure variation according to the percentage of filler materials

To develop some reduced swelling pressures, in the case of desensitization with granular material is recommended that the humidity of the material should have a moisture content with 1...3 % higher than the optimal humidity of compaction ($w = w_{oc}$ + 1...3 %) (Ivasuc, 2013, Olinic et al., 2014).

Laboratory tests revealed that the following mixtures were optimal mixtures in relation to the natural sample: 40% slag, 50% sand with gravel and 30% gravel (Table 5).

Table 5. Soil characteristics of optimal mixtures with granular materials

Material / characteristics of compacted samples	Optimal parameters of compaction		Permeability coefficient	Compressibility characteristics		Shearing characteristics	
	ρ_d^{max} [g/cm3]	w_{opt} [%]	k [cm/s]	$E_{oed200-300}$ [kPa]	p_u [kPa]	Φ [°]	c [kPa]
Natural clay (C)	1.62	21.0	$8.22*10^{-9}$	8403	200	24	43
60% C + 40% S	1.77	14.5	$7.63*10^{-9}$	11905	110	27	35
50% C + 50% SG	1.93	11.2	$1.07*10^{-8}$	11111	110	33	17
70% C + 30% G	1.87	11.9	$4.89*10^{-8}$	7380	75	*25	*35

7.1. Destructuration of stabilized clay samples subjected to temperature and humidity variations

It is mandatory for designers to take into account the hydration-dehydration and freeze-thaw behavior of soils to select proper materials for constructing embankments exposed to temperature and humidity variations.

In order to explain the solution chosen for the optimal mixtures with granular materials were chosen representative samples of clay and mixtures between clay and non-cohesive material (sample 1 – Natural clay, sample 2 – 60% C+40% S, sample 3 – 50% C + 50% SG, sample 4 – 70% C + 30% G) which was submitted to hydration-dehydration and to freezing-thawing cycles from October 2013 until April 2014 (Olinic et al., 2014).

Samples were subjected to a dehydration cycle by drying in normal conditions of temperature and humidity. The phases of drying may be seen in Figure 18.

Figure 18. Clay samples dehydration

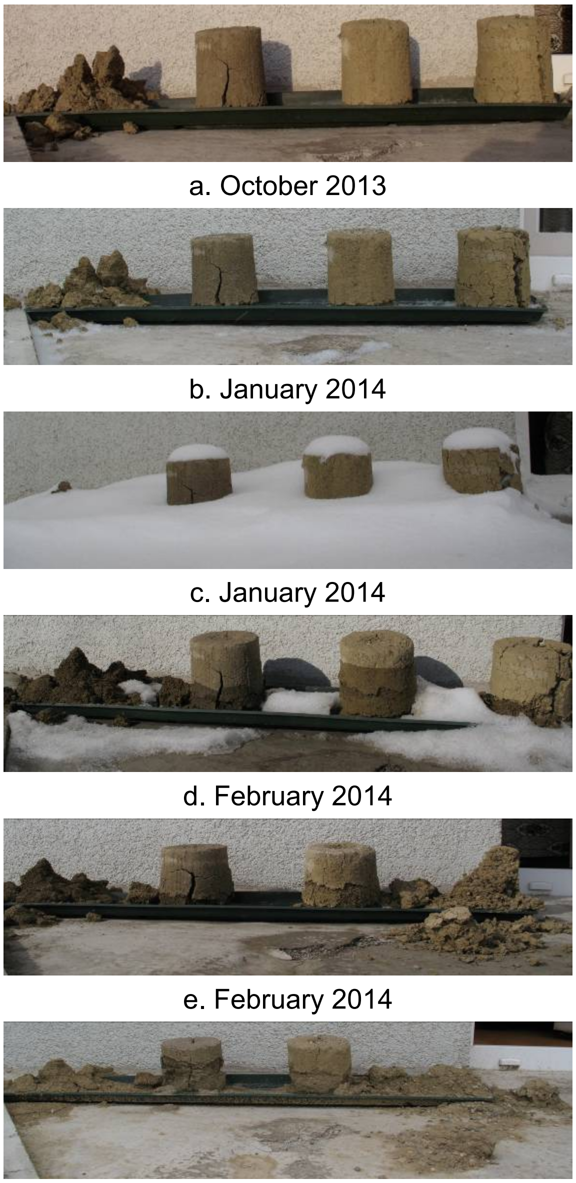
In order to observe the effect of swelling when the expansive clay is in contact to water the samples were flooded at the bottom (hydration cycle - Figure 19) and subjected to drying (Figure 20).

Figure 19. Clay samples flooded at the bottom

Figure 20. Clay samples dehydration after flooding

From October 2013 to April 2014 the samples were exposed to natural weather conditions, being subjected to several freezing - thawing cycles. The samples destructuration effect can be seen in Figure 21.

The simulations reveal the influence that destructuration has on the behavior of an unprotected embankment constructed from expansive clays or stabilized clays after a number of hydration-dehydration and freezing-thawing cycles. It can be noticed that minimum differences of destructuration is observed for the samples from the middle, samples which are represented by the mixture of 40% slag and 50% sand with gravel.



a. October 2013

b. January 2014

c. January 2014

d. February 2014

e. February 2014

f. April 2014

Figure 21. Clay destructuration at different periods of time

Even the sample composed of 30% gravel (sample 4) which developed the lowest swelling pressure value (75 kPa) was destructured, after the several freeze - thaw cycles, as much as the natural sample.

8. CONCLUSIONS

In completion to the European technical norms, Romania elaborated several technical norms and guides for the local specific site conditions.

All the Romanian technical norms are accordingly to the Eurocode. All the European Norms are fully applied in Romania.

Significant areas of the Romanian territory are covered by soils classified as “difficult foundation soils”.

The paper is focused on the stabilization of loessoid and expansive soils by mixing them with different percentages of granular materials in order to improve their engineering properties to use them as soil fillings or foundation soils. On the basis of the laboratory tests and field researches, the following conclusions can be drawn:

- Each soil acts differently depending on its mineralogical and granulometric composition: for this reason there is no ‘recipe’ for the improvement of difficult soils;
- Adding granular materials, for some addition percentages, the effect of compaction is higher that the desensitization effect, resulting in materials considered even ‘worse’ than the natural sample;
- The mixture of loess and granular material has better mechanical characteristics and reduced permeability compared to the one the loess has in its natural state. From all the solutions proposed (compacted loess, mixture of loess and sand and mixture of loess, sand and bentonite) the one with sand and bentonite, mixed with loess after drying, seems to be the optimal one due to the wide domain in which optimal compaction parameters are reached;
- Concerning mechanical characteristics, no significant differences seem to exist between the analysed mixtures, but one can notice that water sensitivity is significantly reduced and that, compared

to the flooded loess, the values obtained are significantly better;

• In the case of expansive soil mixtures with granular materials, the swelling pressure is the only property that changes and also categorizes the activity of a soil in contact with water;

• In order to develop some reduced swelling pressures, in the case of desensitization with granular materials, it is recommended to assure some compaction degrees of 95-98% to a compaction moisture content 1...3% higher than the optimum moisture content.

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