Analysis of the time–shrinkage evolution for an expansive clayey soil

H. Ejjaaouani
Dr sc , Laboratoire Public d'Essais et d'Etudes. Casablanca, Maroc.

J.P. Magnan
Dr sc , Institut français des sciences et technologies des transports, de l'aménagement et des réseaux.. Paris, France.

V. Shakhirev
Dr sc , Laboratoire Public d'Essais et d'Etudes. Casablanca, Maroc.

S.Bensallam
Phd student , Ecole Mohammadia d'ingénieurs – UM5 Agdal. Rabat, Maroc.

KEYWORDS : Clay, swelling, shrinkage, amplitude, analytical model.

ABSTRACT: This article considers a real scall results of a shallow foundation loading tests, carried out in-situ on an expansive clayey soil from its natural water content to a saturation state. The loading and unloading results obtained by these tests were used to evaluate the evolution of the soil deformation properties on the basis of its water content. Furthermore, we used the soil mechanical parameters to analyze the soil resistance according to its water content. Assuming that the soil shrinkage is closely related: to its capacity to distribute the constraints in the soil mass, to its state of moisture and to its physical and mechanical parameters; we elaborated an analytical equation in the aim to calculate the soil's shrinkage amplitude in drying time.

1 INTRODUCTION

Basically, the shrinkage is explained at first by the soil exposure to the air. Indeed, even if there are any external load, internal mechanical and chemical process influence the soil's stability by weakening and/or destroying the skeleton structure.

During the drying process an additional energy appears in the soil skeleton, because of the major reduction of the quantity of water linked to the clay particles surfaces and inside their interfoliaceous spaces. This energy can be preserved in the soil skeleton thanks to the physical and chemical structure reinforcement by the re-establishment of the structural connections between particles (and also their cementing), passing therefore to a new compactness state. The re-establishment of the structural connections happens thanks to the progressive crystallization of minerals in the interstitial salt solution (carbonates, gypsum,…). This new steady state contribute to obtain the shrinkage limit before the total loss of moisture.

In general, the shrinkage limit is reached after the total evaporation of all the free and capillary water and also the slightly linked water (Guilder 1956, Berezantsev 1961, Goldshtein 1971). However, the shrinkage limit in-situ will be different from the value determined at the laboratory.

During a partial drying, the shrinkage limit of the surface layer will corresponds to the value of the water content in the subjacent layer; and that can be explained by the soil perpetual research for a steady state. In this case the moisture and the constraints distribution in the soil will be depth depending, this factors complicates the resolution of the problems related to the calculation of the foundations in expansive soil.

A real-scale experimentation of loading and unloading shallow foundations was performed in the field (Ouarzazate. Morocco) on expansive clayey soil; by the LPEE in collaboration with the LCPC. The aim of this experimentation is to measure and to follow-up the behavior of the shallow foundations putting in contact with the expansive layer during wetting-drying cycle. The experimental devices were described by Ejjaaouani et al. 2008.

A part of this article will be devoted to analysing of the shallow foundations loading tests at different moments of wetting-drying cycle. And in order to resolve the problem of the soil resistance which has different capacity of constraints diffusion (which cause different deformations in the soil below the foundations), we propose an analytical model to calculate the shrinkage amplitude in time.

2 THE EXPERIMENTAL AREA AND TESTS

The experimental area is located in the northwest part of Ouarzazate (Morocco) and the different horizons of the soil were identified by tests pits. From the soil surface we have:
1/ 0-0.5m: gray color gravelly silt;
2/ 0.5-1.8m: slightly sandy compact clay of ocher color;
3/ 1.8-2.9m: slightly sandy indurate clay of yellow to red color;
4/ 2.9-3.5m: sandy clay and clayey sand with yellowish gypsum debris;
3.5-4.5m: compact clay of green color scales disintegrating;
4.5-9.8m: sandy clay of red color.

Four shallows foundations were built in the excavated zone and have being used at different moments;

- The first Shallow Foundation FS-1 was charged twice: before the soil's wetting in order to test its behavior in a natural state (W=12.5%), and after a total saturation (W=35% the average water content under the foundation)
- The foundation FS-2 was tested after the wetting period's end, when the average water content was 27%.
- The foundation FS-3 was tested when the average water content was 20%.
- The foundation FS-4 was tested when the average water content was 16%.

The loading of foundations was performed by a hydraulic jack of 380cm² surface. The pressure was gradually applied step by step in few minutes on each foundation. The forces applied successively are: Q1=76KN, Q2=152KN, Q3=228KN,…. , Q10=769KN.

The foundations settlement was measured in time for each load, until the total stabilization of the displacement. The limit used to define the stabilization of the settlement was 50µm/hour.
Between the first loading of FS-1 and the second one on wet soil, the foundation had lift 90mm.

Figure 1. Measurement devices of the vertical displacements.

Figure 2 below presents two loading and unloading curves of FS-1 at different moistening states. We note that these two curves have the same shape, and if we multiply the horizontal scale of the settlement curve after humidification by 7.6, we obtains a perfect coincidence with the curve obtained for the natural state soil. Thus, after wetting the soil becames 7.6 times more deformable than the natural state soil; in the opposite the unloading deformations are 2.5 times greater.

The table. I below shows the values of the foundations displacements under load according to the water content and the thickness of the active horizon.

Table. I: Water content and vertical displacement values during the loading tests.

Foundation	Test date	Water content (at -20cm)	Free swelling (mm)	Free settlement (mm)	Active zone (m)
FS-1	October 2002	12.54 %	-	-	1.7
FS-1	July 2003	35 %	93	-	1.7
FS-2	September 2003	27 %	76	4 – 6	1.6
FS-3	March 2004	20 %	87	20 – 29	1.5
FS-4	September 2004	16 %	80	35 - 50	1.45

Intact soil specimens were collected during the drying phase at 20cm depth to determine the shear strength parameters according to the water content. These tests have given the values summarized in table II.

Table II. Variation of the internal friction angle according to the water content.

Water content W (%)	12.5	16	20	27	35
Internal friction angle ϕ	35	30	27	26	24

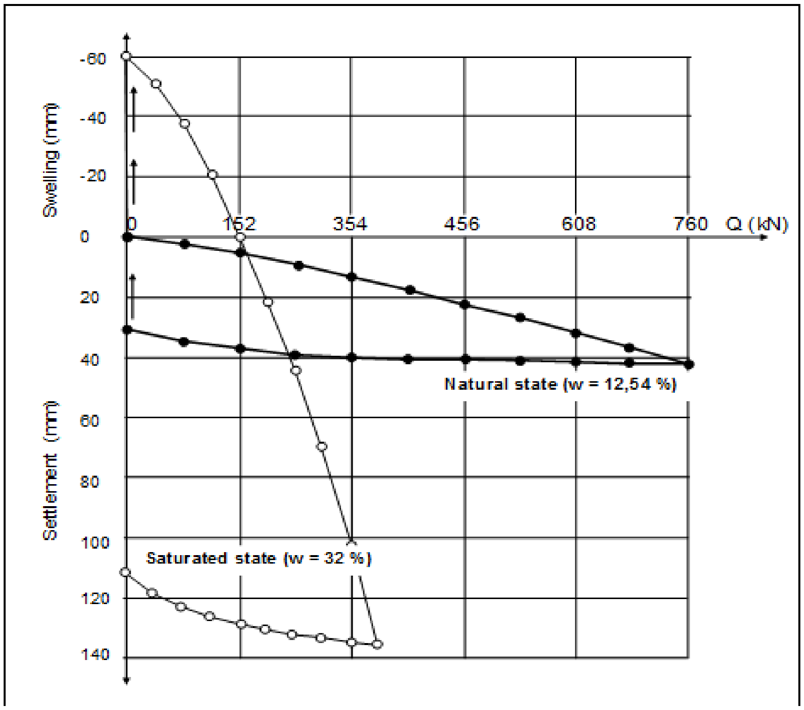

Figure 2. Vertical displacements of the foundation FS-1 during the steps

loading, carried out before and after the soil's humidification.

The shallow foundations loading curves are shown in figure 3 below; some curves was extrapolated to σa = 500kPa. These tests were carried out during the drying process after stopping the both water supplying. so, these results correspond to a decreasing water content distribution; and the soil water contents were taken at 20 cm depth from the surface of the tested clay layer.

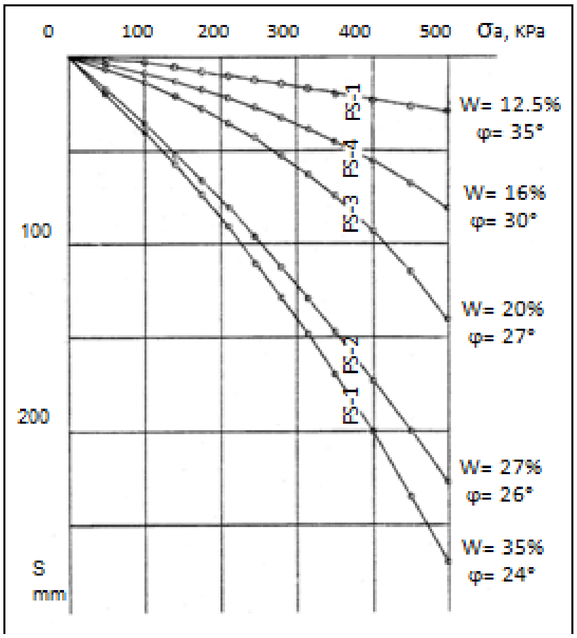

Figure 3. Vertical displacements of the foundations FS-1, FS-2, FS-3 and FS-4 during the loading tests.

For saturated soils since the water content exceeds the maximum adsorption capacity, the displacement of the weakly linked water is impossible; in this case each particle retains its maximum quantity of water and the electrokinetic potential at the diffuse layer surface will be zero as well as the potential difference. Similarly, there is no displacement of water if the soil water content is equal to the maximum hygroscopic moisture content, because the mutual attraction forces of the particle and the hydrated layer is always greater than the mutual attraction forces of the closest particles. In case of an interstitial water with dissolved elements, it may happen a water displacement between diffuse layers of equal thickness until reaching an equilibrium degree of dissolved minerals saturation. When the medium conditions allow it and when the pore solution is fairly concentrated, the dissolved minerals can crystallize and cement the closest particles during drying phase.

At hot weather, it can be a delay for the diffuse layers re-establishment, which lose by evaporation the weakly linked water and recovers it at night when the temperature falls according to the air humidity and the wetter subjacent layers. This re-establishment process continues until all the moisture reserves located in the lower layers run out; after this begins a slow decline in the surface layers water content.

At the beginning of the in-situ tests, there was an intense water content decrease from 35% to 27% due to the evaporation of the free water, this reduction continued for 30 days accompanied by a settlement from 4 to 6 mm through the area test. Then the water content and the settlement was stabilized for 6 months, so from this results we esteem that the quantity of the linked water in this soil is approximately 27%. After an intensive drying of some samples, the measured water content was 4%, which is equivalent to the strongly linked water content.

When the soil still in contact with the air, it always tends to find a new steady state. They usually adsorb or lose water depending on the medium pressure and temperature.

From the previous results, it can be concluded that:

- The quantity of the strongly linked water is 4%;
- The quantity of the weakly linked water is 27-4 = 23%;
- The quantity of the interstitial free water is 35-27 = 8%.

We note that the quantity of the strongly linked water (4%) represents the quantity of water in the hydrated layer which is itself a part of the diffuse layer (according to Lebedev.A.F who esteems that the hygroscopic water is an integral part of the strongly linked water). Also the thickness of the diffuse layer is variable and closely related to the ambient condition.

Note: the values estimated above are approximate, nevertheless it gives a quantitative estimation about the different kind of water present in the soil.

3 SHRINKAGE CALCULATION

Conventionally, the shrinkage limit value is the water content where the soil skeleton recovers its structural links, leading to its strengthening and preventing its shrinkage. The soil shrinkage happens due to a moisture content decreases as a result of the aeration process. Furthermore, the macroscopic effects of this process are closely related to the minerals rigidity, indeed, minerals with mobile crystal lattice have a greater ability to shrink compared to minerals with a rigid crystal lattice. Similarly, non rigid soil with colloidal particles have a high swelling-shrinking potential compared to the soil with large particles. Also, the existence of trivalent cations favors the shrinkage more than the monovalent cations. The soil shrinkage also depends on the aeration temperature, according to Sorochan the most effective temperature which favors the shrinkage process is between 30 and 60 °C.

The increase of the capillary forces contribute to a reduction in the soil volume. This occurs in clay soil when moisture adsorbed by the soil is concentrated at the bottom of the capillary pores and retains soil particles in a bent state.

The settlement of a clayey layer is influenced by the applied load and the weight of the active overlying horizon. Other parameters also affect the shrinking process as the soil ability to distribute the stresses in the soil mass, the water content, the grain sizes and the soil density. It should be noted that the crystal lattice density is higher than the average density of the soil, that's why the evolution of the deformation over time within the aggregates happens too slowly compared to the adsorption process by the forces of these aggregates. According to Mustafaev.A, that explains why the crystalline swelling starts after the onset wetting, and its stabilization finishes after the total saturation. (The reverse process happens during the drying phase).

The swelling and shrinking deformations are a time developed process. According to Rebinder.P.A, the adsorption is not a reversible process, in fact the adsorption and the desorption isotherms do not follows the same path.

Assuming that the soil shrinkage is closely related: to its capacity to distribute the constraints in the soil mass, to its state of moisture and to its physical and mechanical parameters. By using the laboratory and in-situ tests and on the basis of the limit states of the soil behavior; we developed an analytical equation allowing to calculate the settlement amplitude over time and soil moisture. This equation integrates the following parameters: (1) the foundation applied load, (2) the weight of the soil within the active zone, (3) the soil deformability, (4) a sinking coefficient of the soil shrinkage, (5) and the drying degree of the active zone. This equation presents a mixed solution for plastic and elastic resistance in a soil mass with a variable capacity of constraints dispersion.

$$Sr = \Delta hg - \left[\frac{a.b.\sigma_a}{(a+2ztg\beta)(b+2ztg\beta)} + \frac{W_m - W_i}{W_l}\rho z\right]\frac{(1-\nu^2)}{E} z \left(\frac{W_m - W_i}{W_l}\right)\left(\frac{t_i}{t_m}\right)^{\frac{W_m - W_i}{W_m}} \qquad (1)$$

Δhg: soil free swelling at water content W_m ;
W_m : maximum water content at the surface;
a and b: foundation length and width;
σ_a : applied load;
β: dispersion angle of the constraints in the soil mass;

$$\beta = \varphi\left(\frac{Z_i}{Z_{pl}}\right)^{e\left(1-\frac{Z_i}{Z_{pl}}\right)^{e}} \tag{2}$$

φ: internal friction angle;
e: void ratio;
Z_{pl} : soil plastic zone below the shallow foundation;

$$Z_{pl} = \frac{\sigma_a(1-sin\varphi)-2C.cos\varphi}{\rho(1+sin\varphi)}\ exp^{-\pi tg\varphi} \tag{3}$$

Z_i : concerned depth $0 \leq Z_i \leq Z_{pl}$;
C: soil cohesion;
W_i: variable water content;
W_l: shrinkage limit;
ρ: soil density;
Z: shrinking active zone;
E: deformation modulus;
t_i: variable time;
t_m : drying end time;
ν : Poisson's ratio;

$$\sigma_{xyz} = \frac{a.b.\sigma_a}{(a+2ztg\varphi)(b+2ztg\varphi)} + \frac{W_m-W_i}{W_l}\rho z \tag{4}$$

Note that in the equations (3) and (4) the absence of e, ρ and β = φ describes a continuous medium and when the physical and the mechanical parameters are known the equations 1 and 4 describes a discrete medium.

without load (σ_a = 0), the settlement of the soil free surface is given by the equation (5) :

$$Sr = \Delta hg - \left(\frac{W_m-W_i}{W_l}\right)^2 \rho z^2\, \frac{(1-\nu^2)}{E}\left(\frac{t_i}{t_m}\right)^{\frac{W_m-W_i}{W_m}} \tag{5}$$

To perform our calculations we used the following data, obtained from the in-situ tests:

$\Delta hg = 0.093m$, $W_m = 35\%$, $W_l = 16\%$, $\rho = 18KPa/m$, E=1750KPa .

From the observations and the results collected at the experimental area, the thickness of the layer subjected to the drying process is z = 1.7m. The calculations were performed for different water content W_i ($W_1 = 27\%$, $W_2 = 20\%$, $W_3 = 16\%$) and determined at a depth of 0.2m from the ground surface.

Because the shrinking process has a stepped character, the settlement calculation was performed for each water content value W_i independently. Figure 10 shows the results of the settlement calculation of the soil surface as a function of the drying time t_i , for foundations FS-2, FS-3 and FS-4. The observation of the foundation FS-2 was conducted during a time $t_1 = 90\ days$ when the average soil water content in contact with the foundation was 27%. The observation of foundations and FS-3 FS-4 were carried out respectively in a time $t_2 = 270\ days$ for a water content of $W_2 = 20\%$ and $t_3 = 450\ days$ for a water content of $W_3 = 16\%$.

Table III. surface soil settlement during the drying process.

Foundation	Test date	Water content (at -20 cm)	Swelling (wetting) (mm)	Experimental Settlement (mm)	Calculated Settlement (mm)
FS-1	October 2002	12.5 %	-	-	-
FS-1	July 2003	35 %	93	-	-
FS-2	September 2003	27 - 28 %	76	4.5 – 6	4.55 – 5.94
FS-3	March 2004	18 - 20 %	87	20 – 29	20.9 – 26.84
FS-4	September 2004	14 - 16 %	80	35 – 50	33.5 – 40.96

The comparison of the final computation settlements results is represented in table III above. It should be noted that the dispersion of the results collected in the field can be explained by the three facts: (1) the expansive layer thickness is not uniform under the surface test (1.45m on one side and 1.7m of the other side), (2) the soil heterogeneity and (3) the moisture migration. Nevertheless, we receive a good correlation between the experimental results and the calculated results.

Table IV below shows the final values of the shallow foundations settlement; calculated on the basis of the applied load.

Table IV. Foundations settlement according to the applied load.

Foundation	Time of shrinkage (days)	Water content %	Applied load KPa			
			0	210	350	500
			Foundation settlement (mm)			
FS-2	90	27	6	17.5	25.2	33.4
FS-3	270	20	20.9	41.4	55	69.7
FS-4	450	16	33.5	55.6	70	86

The foundations settlement increase with increasing aeration time and applied load.

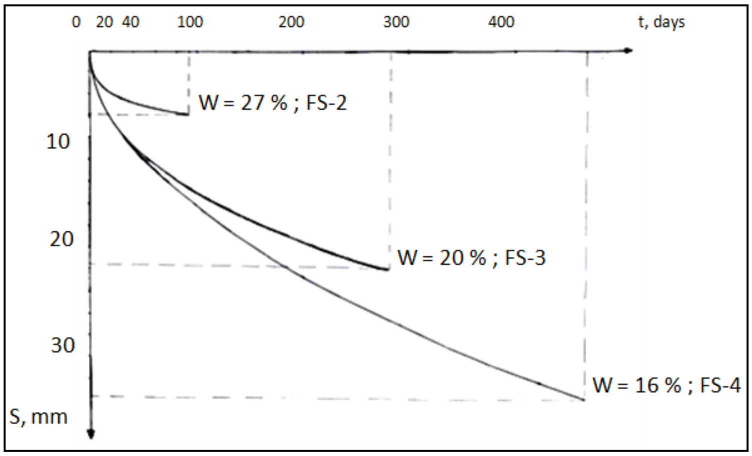

Figure 4. Calculated soil's shrinking curve in time.

At the first drying period's end $t_1 = 90\,days$ the settlement was 6 mm at a shrinking water content $W_R = 27\%$. At the second drying period's end $t_2 = 270\,days$ the settlement was 20.9 mm for $W_R = 20\%$. And at the third drying period's end $t_3 = 450\,days$ the settlement was 33.5 mm for $W_R = 16\%$ which corresponds to the shrinkage limit obtained without load (σa =0) in the laboratory. (Table IV). For the same water content, when the applied load increases the settlement limit also increases, and that can be explained by the destruction of the soil structural strength. In the borderline case where σa = 500kPa, the settlement limit was Sr= 86mm and this value nearly corresponds to the value of the free swelling which is 93 mm, figure 4.

Note: it should be noted that the computation results are approximate, because the special tests in-situ were not carried out.

The preliminary calculations of the soil settlement confirm that due to the moisture decrease on the particles surfaces and inside their interfoliaceous spaces, there will be a new arrangement state of the particles and a possible crystallization of minerals dissolved in the interstitial water (cementing the closest particles). Thus, the soil passes to a new steady state and will be more resistant. In this case, it is take form in the soil different cohesion forces: a coagulation structures as a results of the particles cohesion by the Van Der Waals forces (trough the liquid film), and condensation and crystallization structures by chemical forces of the main valence (Rebinder,1978).
The coagulation structures are characterized by their thixotropy, their mobility and their high elasticity when they are subjected to low resistance, while the condensation structures are fragile, non-thixotropic and have a high resistance.

4 CONCLUSION

During an intense wetting of clayey soil, it happens an increase in volume which leads to the destruction of the fragile structural links between clay particles, leading to the degradation of the soil mechanical properties. When the wetting process ceases, it begins an aeration process (drying process) in the upper soil zone who communicate freely with the atmospheric air. This drying process can also affects the percentage of water content in the subjacent layers which are able to retain water much longer. In the active zone, the weakly linked water always tends to move towards the direction of the higher attraction molecular forces. This displacement of water will continue until the equality of the electric potential between the particles.

The preliminary calculations of the soil settlement confirm that due to the decrease of the moisture on the particles surface and inside their interfoliaceous spaces, there will be a new arrangement state of the particles and a possible crystallization of minerals dissolved in the interstitial water (cementing close particles). Thus, the soil passes to a new steady state and will be more resistant.
The loading tests was performed to analyze the behavior of the shallow foundations during the wetting-drying process. The results representing the vertical displacement of each foundation show that the wetting makes the soil 8 to 10 times more deformable than its original state. and drying process brings back gradually the soil water content to its original state; but the evaporation process happens to slowly and the humidification effects are sustainable.
On the basis of the laboratory and the in-situ tests, we developed an analytical equation allowing to calculate the settlement amplitude over time. The comparison of the final computation settlement results and the experimental data shows a good correlation between the two methods.

REFERENCES

Berszantsev.V.G, Kcenofontov.A.I, Platonov.E.V, Yaroshenko.N.N, 1961. Mécanique des sols et des fondations. Moscou, transzeldorizdat, p 340.
Boumaiz 2003 . thèse Etudes géotechniques des argiles gonflantes de Ouarzazate , Maroc. P,256.
Ejjaaouani.H, 2008. Interactions des fondations et des sols gonflants : pathologie, calculs et études expérimentales. Thèse de doctorat. P 248.
Florin.V.A, 1959. Base de la mécanique des sols. Strojizdat, Moscou, p 360.
Goldshtein.M.N, 1971. Propriétés mécaniques des sols. Moscou, 367p.
Lebedev.A.F, 1936 ; l'eau du terroir et souterraine. Selkhozgiz.
Magnan. J.P, et al 2008. Comportement des fondations superficielles sur sol gonflant lors de l'humidification et du séchage. 2008 Editionsdu LCPC, Paris.
Mustafaev. A, 1989. Les fondations sur sols affaisables et gonflants. Moscou. 578p.
Rebinder.P.A, 1978. Phénomènes superficielles dans un système dispersé, Edition « Science ». Moscou, p 203.
Sorochan.E, 1989. Construction d'ouvrages sur les sols gonflants. Moscou, p 310.