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IDENTIFICATION, ASSESSMENT AND IMPROVEMENT OF COLLAPSIBLE SOILS: CASE OF TUFAS SOILS OF CASABLANCA- MOROCCO

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ABSTRACT

Collapsible soils correspond to metastable soils that can exhibit a large change of volume due to wetting with or without extra loading. Consequently, they are susceptible to cause significant deformations at their saturated state, which represent real challenges to geotechnical professions. In fact, numerous soils can fall in the category of collapsible soils but contrariwise, they can show at the same time high degree of stability and can support heavy loads under their natural water content. The present research aims to suggest a method to conduct when collapsing behavior is detected in materials used during geotechnical projects, based on three parameters: potential of collapse, thickness of layers and stress level. To do so, we conducted experimental work included Atterberg limits, Particle size analysis (sieving and sedimentation) and Value of Methylene blue to a group of five undisturbed specimens obtained in the city of Casablanca. They have approximately similar characteristics: tend to vary from slightly plastic to medium plastic, mostly made of fine grains with a wide distribution of particles and have a small value of fineness modulus. The low dry density, and the high void ratio and stiffness characterizing the five undisturbed tufas samples Had lead us to elaborate an oedometric test without changing the initial water content, and then we had recorded the settlement once the equilibrium reached when the soils were wetted till saturation under a stress of 60 KPa, 100 kPa, 140 kPa, 200 kPa and 240 KPa. Assuming that every infrastructure work has its critical collapse potential that can be admitted, and in light of the oedometric test results, we can improve the serviceability and reduce the cost and the frequency of rehabilitation by either reducing the support section without making any change to the thickness of the layer or deepen the tread with maintaining the one dimensional compression.

Keywords: collapsible soils, water content, potential, collapse, dry density, high void ratio, oedometer test.

1. INTRODUCTION

The collapsible soils correspond to metastable soils that manifest a radical rearrangement of its particles and therefore a significant change in volume as a result of wetting or the presence or not of additional loads. They represent then a real geotechnical problem that threatens the longevity of civil engineering projects. Many of the damages were caused by this type of soil and made headlines in the media. In New Mexico, a differential settlement due to the watering of lands near a shopping center has caused enormous financial losses, while in northeastern China, several provinces have a large number of houses that have become uninhabitable, the appearance of holes on roads, dangerously leaning buildings, or broken pipes caused by the pumping of groundwater that has weakened the soil [1]. The same phenomenon has occurred in Egypt, the United States, Russia, Turkey and several arid to semi-arid countries of the world [2, 3].

To better understand the problem, Barden [4] have determined the main conditions underlying the phenomenon of collapse: (1) an open, partially saturated structure, (2) an existing or additional stress, (3) sufficiently resistant bonding elements to stabilize the inter granular contacts and relatively weak to disappear or be weakened during imbibitions. For further researches, El korshi [5] have demonstrated using the Bishop Wesley triaxle equipped with sensors, that adding water to the soil without any extra loads besides gravity suddenly leads to the appearance of deformations starting from a critical water content, called collapse water content, for which the soil instantaneously loses its mechanical strength with an increase at the same time of the Interstitial pressure due to rapid particle rearrangement. The collapse occurs then due to the phenomenon of coalescence that leads to the diminution of the cohesion. (Figure-1)

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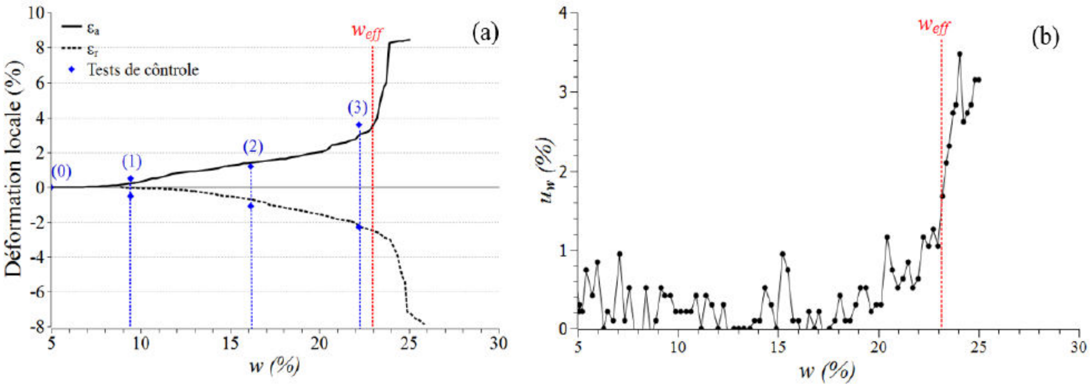
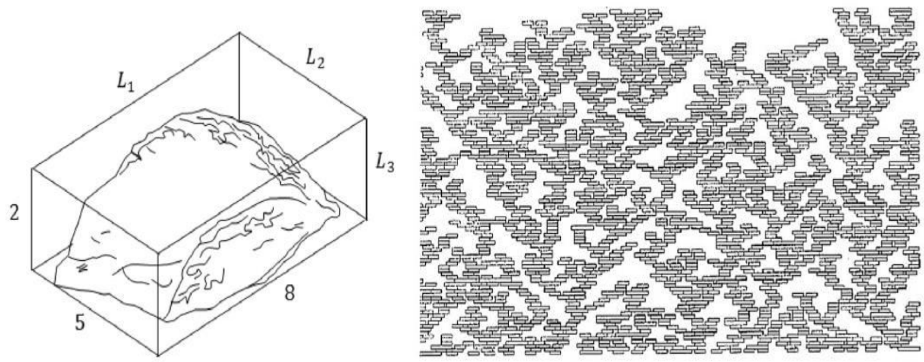


Figure-1. (a) Deformations visualized starting from a critical water content (1) and (b) evolution of interstitial pressures around the critical water content [5]. This test was applied to a model of collapsible soil.

The rearrangement of particles and their relation with cohesion is explained more by Smalley [6] et Dibben [7] who have adopted the Monte Carlo method suggest that loess particles are flat with particle side ratios 8:5:2 then, if two particles overlap by more than the value of critical bonding then attachment will occur, cohesion will develop, otherwise the upper particle will move sideways and fall. (Figure-2)



(a) (b)

Figure-2. (a) flat particle as imagined by Rogers and Smalley [8] and (b) random structure composed by flat particles overlapping [7].

But in the field, especially during construction projects, the problem is more complicated because soils are subjected simultaneously to several constrains and the principal mission of engineers is to deal with soils collapsing under both water ads and extra loads. The present work consists on identifying, assessing and improving collapsible soils taken from the city of Casablanca, using geotechnical laboratory tests.

2. METHODS AND MATERIALS

2.1 Double oedometer test

This test consists on placing two samples in oedometers; we maintain the first one at his natural state, with its initial water content while the second one is inundated. Both samples subject the same stress values. The difference between the compression curves is the amount of deformation that would occur at any stress level at which the soil gets saturated [9].

The collapse potential is calculated based on eq. (1) [10]. The variables $\Delta h_{\sigma d}$ and $\Delta h_{\sigma w}$ (Figure-3) correspond to the deformations of soil samples subjected to the vertical stressσ, $h_{\sigma d}$ and $h_{\sigma w}$represent the thicknesses of dry and wet soil specimens [11].

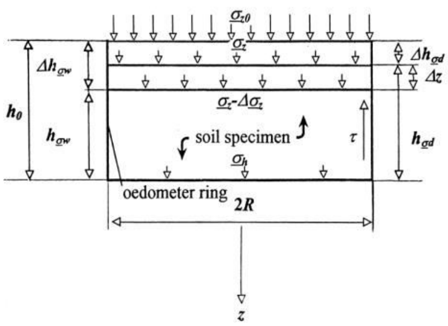


Figure-3. Designations for the double oedometer test used by (Reznik, 2000).

$$\frac{\Delta h}{h_0} = \frac{1 - \ln[(1+m\sigma_{zo})/(1+m\sigma_h)]/[m(\sigma_{zo}-\sigma_h)]}{2mE_0} \quad (1)$$

and

$$\sigma h = \sigma z0 - \frac{2h\tau}{R} \qquad (2)$$

Where:

σz0 : stress applied to the specimen at z=0
σh : stress applied to the specimen at z=h
M : coefficient included in equation (2)
Eo : initial deformation modulus
Ho : soil specimen initial thickness
Δh : deformation value of the specimen
R : Oedometer ring radius
τ : Lateral friction force

After a series of calculations [11] the equation (1) would become at the saturated state:

$$\frac{\Delta h}{h_0} = \left(\frac{e_0 - e_\sigma}{1+e_0}\right)_w \qquad (3)$$

And at the dry state (1) would be written as:

$$\frac{\Delta h}{h_0} = \left(\frac{e_0 - e_\sigma}{1+e_0}\right)_d \qquad (4)$$

Potential of collapse is then written using (3) and (4) [11]:

$$Cp = \frac{\Delta h}{h_0} = \left(\frac{e_0 - e_\sigma}{1+e_0}\right)_w - \left(\frac{e_0 - e_\sigma}{1+e_0}\right)_d \qquad (5)$$

According to Reznik [11], the value of Cp depends on the porosity of the both specimens and the external stresses: in double oedometer test, specimens must have the same porosity which is not possible to achieve, and the equal external stresses don't predetermine equal energy losses between the oedometer rings and soil samples surfaces. Cp will then make engineers take unnecessary and expensive measures to prevent damages that may not happen as how they may be expected. Booth Delage and Noorany [12, 13, 14] have demonstrated that using the method of double oedometer may overestimate the collapse potential up to 10%. This is why single oedometer test might be more useful in this stage of study.

2.2 Classic Single oedometer test

The oedometer test allows the geotechnical risks associated with soils to be identified by describing the amplitude and speed of soil compaction. The test system has two parts:

- A cell containing the soil sample
- The loading system: The loading can be by weight or take a form of pneumatic or hydraulic loads

The principle consists in applying a constant load to the sample and measuring the corresponding settlement and afterwards higher loads are applied. The interpretation consists in plotting the variation curve of void ratio as a function of applied stresses. (Table-1)

Table-1. The different stages of the single oedometer test.

Stage 1	Stage 2	Stage 3	Stage 4	Stage 5
•Specimen placement	•Wetting specimen till saturation	•Application of loads in stages and measurement of settlement after 24h	•Déchargement de l'éprouvette et détermination de la variation de l'indice des vides	•Unloading of the specimen and determination of the variation of the void index

Five specimens (A), (B), (C), (D) and (E) were selected from the city of Casablanca located in different places and have performed the same geotechnical tests as explained in our previous work [15].

Tufas soils specimens are chosen for this present work because they are very predominant the metropolis of Casablanca and many important projects are implemented on this type of soils. (Figure-4)

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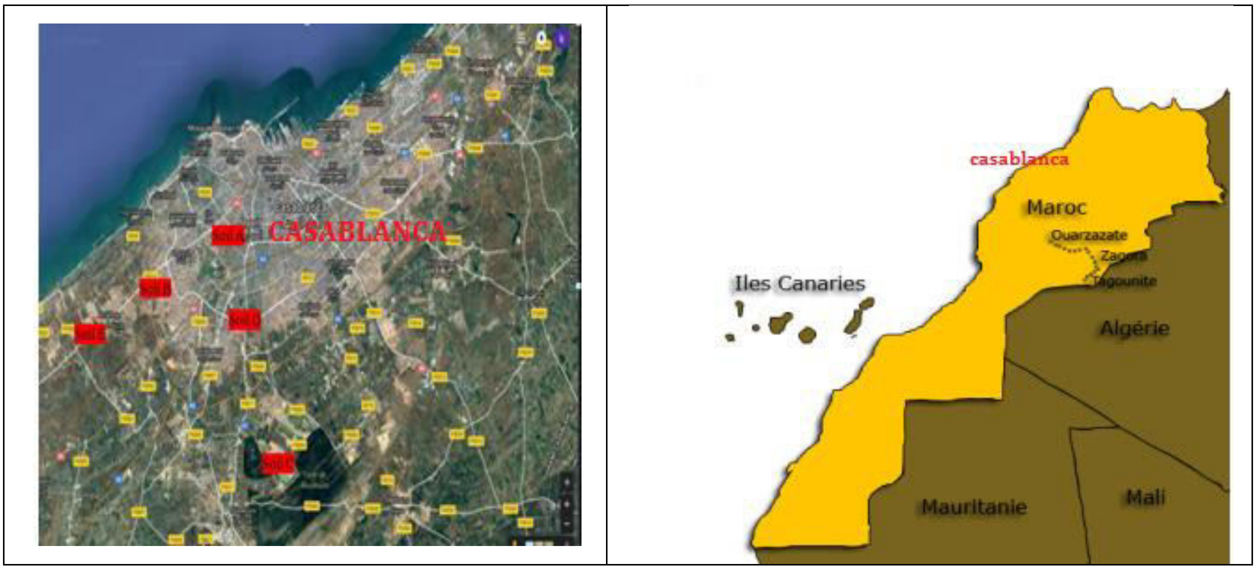

Figure-4. Soil samples from the city of Casablanca located as: Soil A: x=33.545718, y= -7650180, Soil B: x=33.533538, y= -7.709063, Soil C: x= 33.475052, y= -7.610840, Soil D: x= 33.526701, y= -7.741550, Soil E: x= 33.527637, y= -7.741550 (Google map 2017).

3. EXPERIMENTAL WORK

3.1. Tests

We conducted a protocol of experiments based on Norms as specified above [15]:

Atterberg limits	**NFP 94052-1 and NFP 94-051**
Value of Methylene Blue	NFP 94-068
Particle size analysis by sieving	NM 13 1-008
Particle size analysis by sedimentation	NFP 94-057

Samples selected present the same characteristics:

- They tend to vary from slightly plastic to medium plastic
- They are mostly made of fine grains
- The value of fineness modulus is close to zero. It is an empirical factor used in particle size analysis by sieving, obtained by adding the total percentages of a sample of the aggregate retained on each of a specific series of sieves; 0.16 mm - 0.315 mm - 0.63 mm - 1.25 mm - 2.5 mm - 5 mm - 10 mm, and dividing the sum by 100. The fineness modulus Mf, when it is near to zero reveals a fine grain distribution [15].

The following tests are performed to all samples, but to simplify our analysis, we will show only the results of sample (D) but the settlements of each one of them, including the average collapse potential are reported in Figure-8 and Table-2.

To perform the classic oedometer test, we submitted sample D to this test and then plotted the corresponding compressibility curve. (Figure-5)

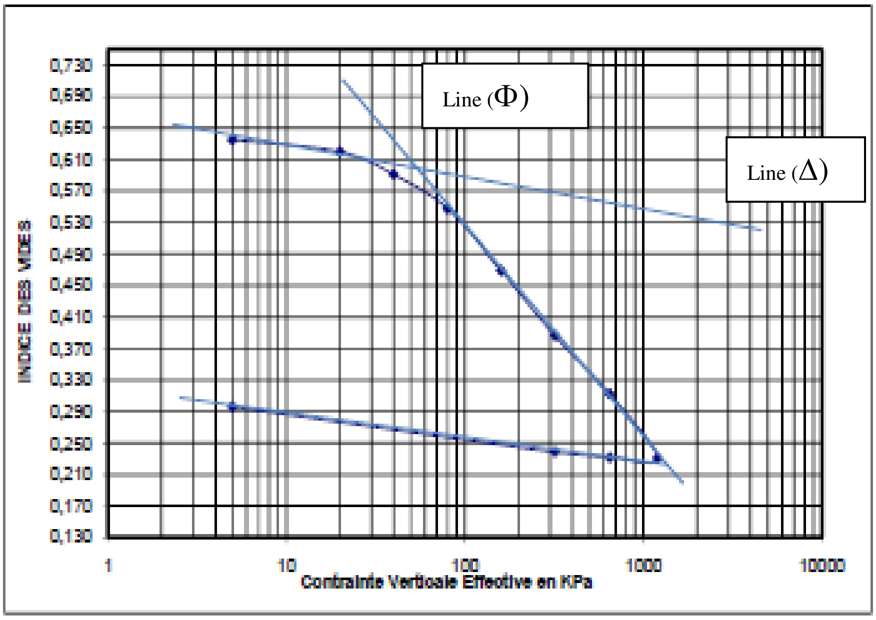

Figure-5. Oedometer results of specimen (D).

Sample (D) has a density of 2.7, initial water content 5% and the initial void ratio: 0.635From the shape of the curves we can determine the following parameters:

- The swelling index C_S which is the slope of the line (Δ). Graphically, C_S= 0.014
- The compression index C_c is the slope of the line (Φ), graphically C_c= 0.3
- Pre-consolidation pressure σ' Corresponds to the point of intersection of the straight lines (Δ) and (Φ) Graphically $\sigma' = 55$ KPa

The results obtained graphically lead to deduct that the soil (D) is highly compressible. The conventional oedometer test is carried out on soils after saturation, which is not the objective of our study which is the evaluation of the behavior of unsaturated soils. In fact, we try to subject the sample to a law of behavior that takes into account the geotechnical parameters of the soils, and more particularly the initial water content.

3.2. Oedometer test redesigned

For each sample we performed the oedometer test as follows: The samples are placed inside a non-deformable cylindrical enclosure, the inner wall is in direct contact with the specimen. We assume that during the oedometric test, the deformations are one-dimensional, which amounts to considering that the lateral deformations are not considered.

During this test, the sample height is 2cm, the oedometer section is 39.80 cm² and the dry soil mass is around 131.01g. (Figure-6)

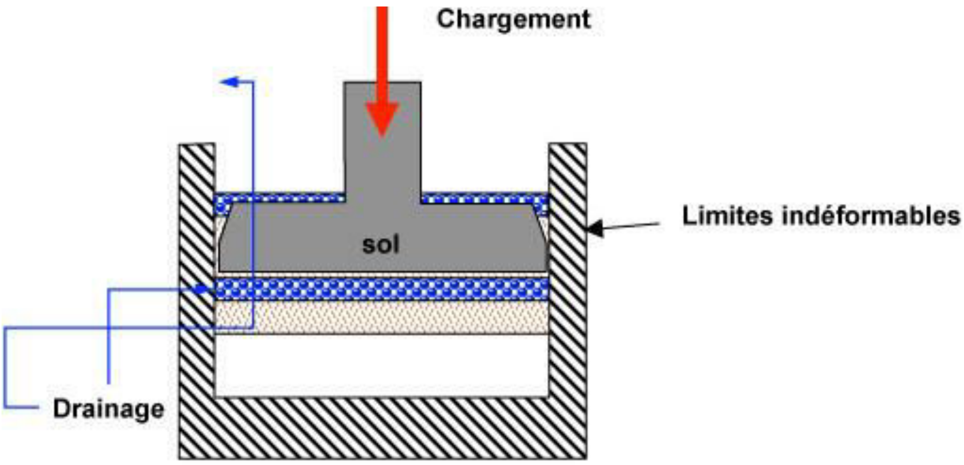

Figure-6. Oedometer test model.

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The initial water content of the sample is maintained and we applied a constant compressing forceσ.

The sample is then inundated under this load after stabilization. Only one collapse value is determined during the performance of this test under the stress valueσ. Therefore, each of the five specimens undergoes constant loads corresponding respectively to: 60 KPa, 100 KPa, 140 KPa, 200 KPa, 240 KPa, in order to obtain complete information about deformations and their relations with stress values. The collapse potential is calculated as follows:

$$Cp = \frac{\Delta Hi}{H} \quad (6)$$

With ΔHi specimen settlement corresponding to the vertical stress σi and H soil specimen initial thickness.

4. RESULTS

Oedometer test performed on specimen D is reported in the graphs below showing the relation between the settlements in terms of vertical stress value. (Figure-7).

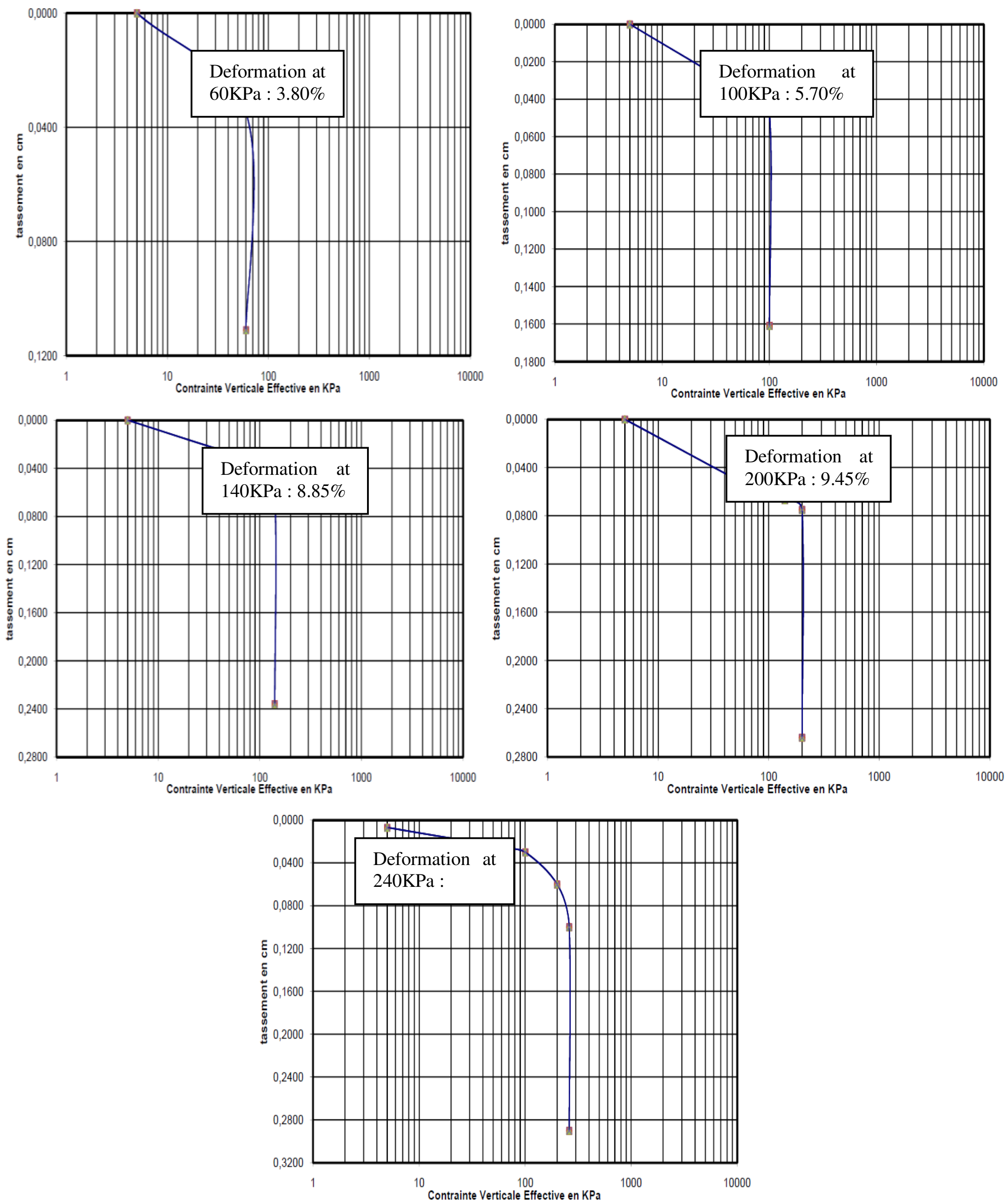

Figure-7. Deformations of soil (D) under vertical stresses: 60 KPa, 100 KPa, 140 KPa, 200 KPa, 240 KPa.

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The samples taken from this study are calcareous tufas with fine particle size. The inter-granular bond is almost due to the presence of capillary bridges connecting the particles. At an initial water content close to 5%, and during wetting under a constant load, the deformation increases linearly from 3.80% under a pressure of 60KPa and reaches 9.45% for a stress value of 240KPa. The amplitude of collapse is then as much important as the vertical stress is significant. On the practical level, for a layer of tufas at a thickness of four meters, a foot applying a pressure of 200 KPa would have triggered a settlement of 37.8 cm. By performing the same test on the other samples and calculating the average collapse potential, (Table-2) we report the results as below (Figure-8).

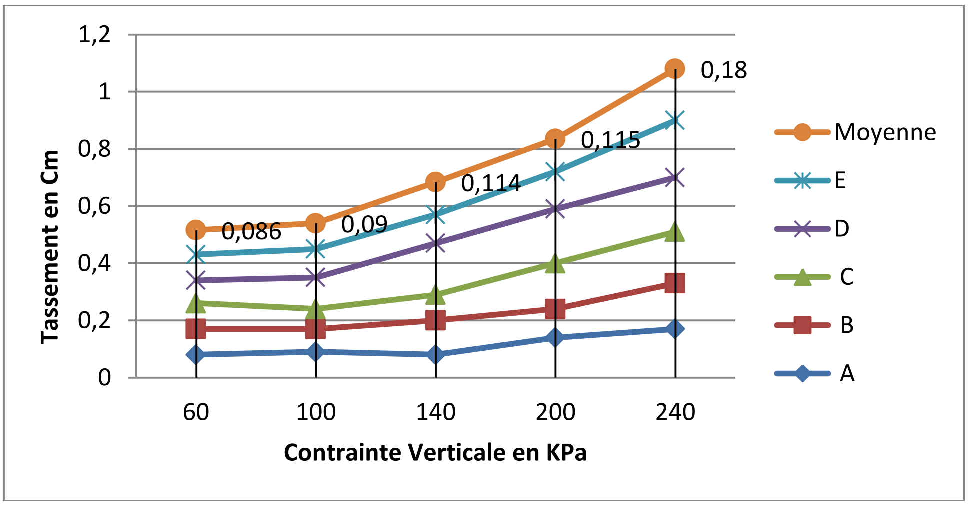

Figure-8. Settlements of soils (A), (B), (C), (D) and (E) under stresses 60 KPa, 100 KPa, 140 KPa, 200 KPa, 240 KPa.

Table-2. Average potential collapse of soils (A), (B), (C), (D) and (E) under stresses 60 KPa, 100 KPa, 140 KPa, 200 KPa, 240 KPa.

	60 KPa	**100 KPa**	**140 KPa**	**200 KPa**	**240 KPa**
Cp (moy)	4.3%	4.5%	5.7%	5.75%	9%

Graphically, the curve of the settlement increases with the stress applied, the capillary bridges connecting the particles of the fine soils are much more numerous than those with large particles. Consequently, the destruction of these bonds requires a minimum volume, unlike soils of coarse granularity which undergo an important gravity effect, which makes them insensitive to the presence of water.

5. DISCUSSIONS

The main problem of soils that might fall in the category of instable soils is the absence of universal norms during geotechnical studies delivering quantitative and qualitative appreciation of the collapse. Numerous authors [16, 17, 18, 19] have found multiple criteria based on index plasticity, saturation degree, void ratio or dry volumetric weight.

Jenning and Knight [20] have come with the oedometer method used by most authors in this field. The test suggests calculating the potential collapse at the stress value of 200 KPa. The specimen must be inundated at that level. The equation (6) used to calculate potential of collapse of the specimens (A), (B), (C), (D) and (E) at the constrain 200 KPa shows that tufas soils are collapsible, according to the measurement of collapsible potential soils [21] (Table-4)

Table-3. Degree of collapse.

Collapse potential (%)	**Collapse degree**
0	None
0.1 - 2	Small
2.1 - 6	Moderate
6.1 - 10	Slightly moderate
>10	Severe

This method used in the present work does not show the other aspect of the liaison between vertical stress and the collapse potential. In fact, the more the vertical constrain is important, the higher becomes the soil's density before saturation. This behavior is explained more by Basma and Tuncer [22] but the context of our work is to find a simple method to identify collapsible soils in order to correct them. That was the reason behind subjecting specimens to different stresses and calculating the average potential collapse. In fact, In order to evaluate the collapse of the soils on the basis of the previously calculated Cp, we adopt the following method: We first suppose that each structure possesses a tolerable collapse potential Cp (limit), above which the deformations become more important and require corrections. In this example, deformations less than Cp (limit) are allowed and are denoted "A" otherwise we note the symbol "NA": Not Admitted (Table-5)

Table-4. Methodology of correcting soils.

	Thickness of layer: 1m	Thickness of layer: 2 m	Thickness of layer: 3m	....	Thickness of layer: n m
60 KPa	A	A			
100 KPa	A	A 1			
140 KPa	A	NA			
…		2			
n KPa					

If A is an n by n matrix corresponding to the Table-4:

$$A=\begin{pmatrix} A_{11} & \cdots & A_{1n} \\ \vdots & \ddots & \vdots \\ A_{n1} & \cdots & A_{nn} \end{pmatrix}$$

Three scenarios are then possible:

- If A_{11} > Cp (limit) the soil should be replaced.
  m and p Two natural integers superior strictly to 1:
- If A_{mp}< Cp (limit) then the soils is admitted
- If A_{mp} > Cp (limit) then A_{mp} is not admitted. (NA) In case where m=3 and p=2 (see table). To return to the state where the deformation is less than Cp (limit), either we maintain the same layer of 2m but we will decrease the vertical stress which leads us to change dimensions of the support section (way 1) as below:

If σ and S correspond respectively to the vertical stress and the support section of A_{32}, σ′ and S′ the vertical stress and the support section of A_{22}

To get a value of A_{32}bearable, we should find a new section S′as:

$$S' = \frac{\sigma}{\sigma'} S \qquad (7)$$

Or we will keep the vertical stress and deepen the foot. (Way 2)

6. SUMMARY AND CONCLUSION

Geological studies can help to identify the location and the deposit of collapsible soils by providing information about genesis of soils and their characteristics. But geotechnical tests should be performed to confirm either the soils are collapsible or not.

Triaxial test, Atterberg limits, Double and single oedometer besides modelling can be helpful to predict and then to avoid damages caused by these soils. In Morocco, we talk more about expansive soils then collapsible soils. They are both instable soils causing important settlements in roads and constructions. In this present work, we had performed oedometer tests on soils in the city of Casablanca. The results have shown easily how important were the deformations due to the wetting. To overcome the situation, we had suggested first to perform the single oedometer rather than the double oedometer for the reasons mentioned in the introduction and then we had recommended a simple methodology to correct soils in case they are susceptible to collapse.

In terms of modelling, collapsible soils are considered as unsaturated soils. The key issue is to find the boundary values at the unsaturated state, which may incorporate numerical algorithms to solve the problem. This is our next stage of study.

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