On the behavior and identification of collapsible soils

Sur le comportement et l'identification des sols effondres

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ABSTRACT: The objective of this paper is to present research first on the factors that control collapse, which are; the effect of silt content and the surcharge intensity and second; a comparative treatment of major methods of identification.

RÉSUMÉ: Le but de cette etude est de presenter la recherche d'abord des facteurs qui controle l'effondrement, ceux-ci etant l'effet du contenu de sols limoneux et l'intensite de surcharge, et deuxiemement un traitement a comparaison des methodes principales d'identification.

1 GENERAL

1.1 *Introduction*

Structurally metastable soils bear large loads but collapse upon inundation causing excessive settlements. This paper deals with the behaviour and identification of collapsible soils.

1.2 *Theoretical Background*

Collapse is a rearrangement of silt size particles in a metastable soil matrix. The metastable state is a result of the frictional shear strength due to capillary water pressure at the contact points of individual silt grains that disappear upon inundation; causing slippage at these points and hence collapse. Thus collapse, although in itself involves a volume reduction, owes its underlying cause to a loss of shear strength [1]. It is common that silty clays may first show an increase in volume upon inundation for a length of time and they collapse still later. The initial swelling in such cases is due to the swelling of the clay packets presented in the voids. However, when slippage occurs, if the surcharge is high enough to induce shear stresses at the silt grain contact points that is in excess of the strength there, the process is reversed and settlement ensues. Since this phenomenon is a major risk factor in foundation engineering, there have been attempts to identify collapse potential.

2 THE MECHANISM

2.1 *Effect of Surcharge and Silt Content*

Collapse behaviour of silty clays with high content of silt fraction is widely investigated by many researchers [1], [4], [5]. In order to understand the variation of collapse behaviour of silty clays within wide range of Ip values, the present study is focused on rather clayey soils. The investigation was made by utilizing soil samples prepared by mixing Ankara clay [2] with 1,3,5,7 per cent silt [3].

These soil samples are statically compacted in large CBR moulds with water content and the dry density were taken as 25% and 1.56 gr/cm^3. Consolidometer samples are extracted from the moulds, loaded by 0.25; 0.50; 1.00; 2.00; 4.00 kg/cm^2 in the consolidometer and then soaked. Dial readings were taken for the consequent 24 hours.

Hereinafter the term "strain" refers to strain observed in consolidometer device and indicates swell if positively valued and collapse if negatively valued. The samples are then soaked and their volume change is measured. Figure 1. shows strain plotted against the logarithm of surcharge for different Silt Content (Plasticity Index) samples. They almost fall in a family of parallel straight lines.

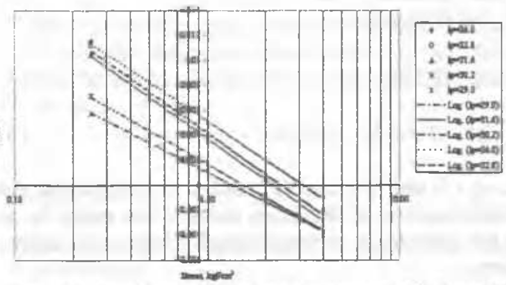

Figure 1.Volume Change-Surcharge Pressure at Different Silt Contents

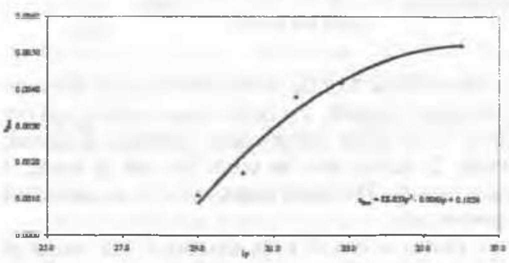

Figure 2.Free Swell versus Ip relationship

The figure above shows that the Swell Pressure (where the lines intersect the zero strain axis) and free swell (at zero surcharge pressure) increase with increasing Plasticity Index as expected. These modes of behaviour are shown in Figure 2 and Figure 3 respectively.

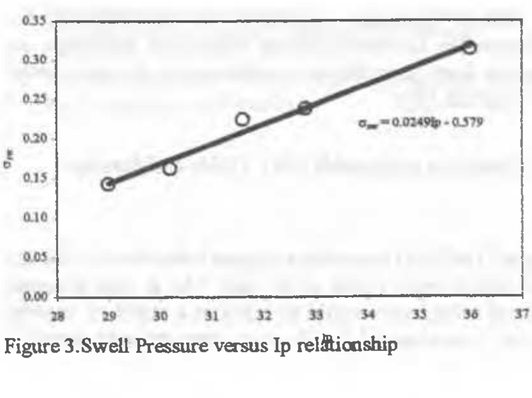

Figure 3.Swell Pressure versus Ip relationship

The following symbols are used in the expressions below:

σ_{sw} - swelling pressure;
m - slope of strain – stress relationship;
ε - strain
s - stress on soil during inundation.
ε_{free} - free swell strain;
Ip - plasticity index, %.
ε - strain to be anticipated to occur.

The Linear relationships in Figure 1 can simply be formulated as below:

$$\varepsilon = \varepsilon_{free} - m \log \sigma \quad (1)$$

$$m = \varepsilon_{free} / \log \sigma_{sw} \quad (2)$$

Substituting (2) into (1) yields :

$$\varepsilon = \varepsilon_{free} - \frac{\varepsilon_{free}}{\log \sigma_{sw}} \cdot \log \sigma \quad (3)$$

Expressions obtained by best-fitting the relationships in Figures 2 and 3, are written as:

$$\sigma_{sw} = 0.0249 \, Ip - 0.579 \quad (4)$$

$$\varepsilon_{free} = 8 \, 10^{-5} \, Ip^2 - 0.006 \, Ip + 0.1036 \quad (5)$$

Substituting (4) and (5) into (3) results in an equation enabling the determination of the strain amount that might be anticipated in the soils for a certain Plasticity Index and under a given surcharge:

$$\varepsilon = (8 \cdot 10^{5} \cdot Ip^2 - 0.006 \cdot Ip + 0.1036) - \frac{(8 \cdot 10^{5} \cdot Ip^2 - 0.006 \cdot Ip + 0.1036)}{Log(0.0249 \cdot Ip - 0.579)} \cdot Log(\sigma) \quad (6)$$

It is to be borne in mind that the initial conditions of the samples used for this study, namely, the initial water content and dry density are likely to be other independent variables of volume change behaviour. It should also be noted that the Ip range is relatively narrow as well. Therefore equation (6) is an indication of behaviour pattern only.

Tsytovich [4] having reviewed large number of test results of his own and other researchers, stated that collapse strain of highly collapsible soils (having low Ip values) can be predicted by using polynomial correlation function which, in certain cases, might be of even first degree. Relying on this Tsytovich finding and summarizing above behaviour of silt mixed Ankara clay, the effect of silt content (Ip) on collapse behaviour silty clays can be described as follows:

The lower silt content (high Ip values) results in logarithmic relationship between collapse strain and surcharge intensity (Fig.1). Within a certain Ip range, the collapse strain of similar soils with the equal dry densities and initial water content might be correlated to Ip values by expressions similar to Equation (6).

As silt content increases (decreasing Ip) the function describing relationship between collapse strain and surcharge intensity changes from logarithmic to polynomial or even linear function. (Tsytovich [4]).

2.2 *Identification of collapsible soils: Gidds and Russian criteria*

Gibbs et al (1967) [5] classifies collapse behaviour by the dry density and liquid limit. Gibbs et al state: "As it was apparent that the critical subsidence would develop as a result of wetting and saturation, it was logical that the basic properties to correlate

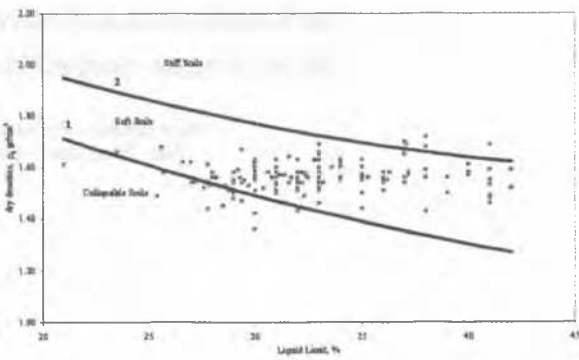

Figure 4. Soil State Criteria. 1-dry densities with void space to hold liquid limit moisture content; 2-dry densities with void space to hold plastic limit moisture content (plotted by using data given in [9]).

with subsidence were density (dense or loose) and liquid limit. For soil near the surface and considering the overburden pressure to be of low significance, the basic criteria used showed that a density with a void space larger than that required to hold the liquid limit moisture content could result in a soil that would be saturated to the point of practically no plasticity and strength and subsequently collapse. Conversely, a soil with a void space of less than the amount for the liquid limit moisture content would always be plastic even when saturated and would not be subject to collapse unless loaded. Therefore the void space required for the liquid limit moisture content was used to compute the limiting density."

Following this logic it may be stated that there is an another limiting state represented by the densities with void space as large as to hold plastic limit moisture content. Then the soils with the higher densities would remain in stiff state when fully saturated and would not collapse to any degree whereas those in between may show plastic flow.

Figure 4 shows both these limiting states applied to Ingushetian silty clay samples [9]. Samples on the right hand side of curve 2 are non-collapsible to any degree. Samples falling in between curves 1 and 2 would soften upon saturation and the samples on the left hand side of curve 1 would exhibit drastic collapse.

One other criterion is that recently (1983) presented in Russian Building Codes [6], which identifies a soil as collapsible if the collapse strain under design surcharge upon inundation exceeds 0.01 [7]. It is straightforward to test this criterion: A collapse test in the consolidometer under the requisite surcharge is done to determine the strain and ascertain that it is within acceptable limits or a double oedometer test is made. The criterion has more practical importance and significance in engineering sense rather than academic implication. In fact, the introduction of such a limiting value is aimed to extend the definition to settlement of soft clays. Presumably the Russian criterion would fall somewhere in between curves 1 and 2 (Fig.4).

To build correlation between Russian and Gibbs criterion, it is assumed that the magnitude of collapse of two soil samples with the same liquid limit would be proportional to the relative distance of its representative point to the curves 1 and 2 on the Dry Density – Liquid Limit plot of Figure 4. If this relative distance is denoted as α, and is given by the following expression

$$\alpha = \frac{\frac{1}{\rho_L} - \frac{1}{\rho_d}}{\frac{1}{\rho_L} - \frac{1}{\rho_p}} \quad (7)$$

and the following symbols are used therein and below:

α - soil state coefficient;
ρ_L - dry density of a soil with the void space as large as to hold
liquid limit moisture content;

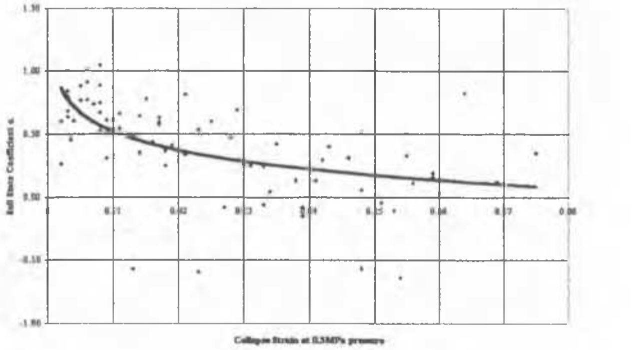

Figure 5. Relationship of Soil State Coefficient α versus Collapse Strain at 0.3MPa Load (plotted by using data given in [9]).

ρ_p - dry density of a soil with the void space as large as to hold plastic limit moisture content;

ρ_d - in-situ dry density of the soil.

ρ_w - density of water;

Then the coefficient α will describe the state of soils upon inundation. Soils with α more than unity would be stiff and its consistency would not be affected by soaking. For α less than 0 soils would be purely collapsible exhibiting sudden collapse settlement whereas the soils with α in between 0 and 1 would soften upon soaking.

Relevant substitutions and simple rearrangement of equation (7) will yield to the following variable criterion for collapsible soils:

$$\rho_d < \frac{\rho_L}{1-\alpha \cdot Ip \cdot \frac{\rho_L}{\rho_w}} \quad (8)$$

Figure 5 shows the relationship of soil state coefficient α of Ingushetian silty clay versus collapse strain observed at 0.3MPa of surcharge intensity as determined by consolidometer collapse tests [9]. As it can be seen from the figure a value corresponding to the collapse strain of 0.01 (Russian criterion) can be conservatively stated as 0.8 to encompass most data.

Russian collapse criterion applied to Ingushetian Silty Clay subjected to 0.30MPa surcharge would be obtained by putting α = 0.8 in Equation 8.

$$\rho_d < \frac{\rho_L}{1-0.8 \cdot Ip \cdot \frac{\rho_L}{\rho_w}} \quad (9)$$

Ingushetian silty clay samples satisfying above criterion (9) can be identified as collapsible to Russian Building Code [7] requirements.

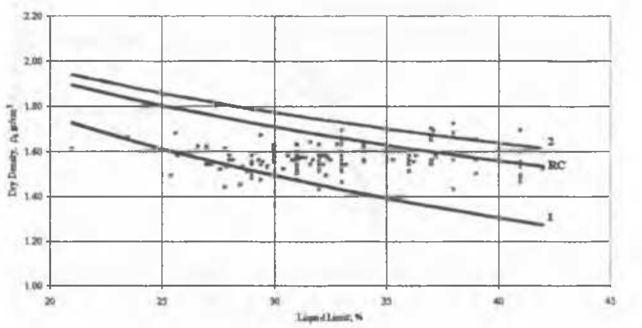

Figure 6. Russian and Gibbs Criteria for Collapsible Soils. 1-dry densities with void space to hold liquid limit moisture content; 2- dry densities with void space to hold plastic limit moisture content; RC-Russian criterion applied to Ingushetian silty clay.

Figure 6 depicts all of the criteria applied for Ingushetian Silty Clay.

Since α coefficient is a property of a soil indicating its behaviour upon soaking and giving an idea of the possible magnitude of collapse, assigning the values of α depending on the structure importance, may help in differentiating the sites with respect to collapse hazard and thereby significantly reduce soil improvement costs.

3. CONCLUSIONS

1. The collapse behaviour is seen to be closely related to the surcharge and the silt content.

2. The behaviour of collapse/swell strain with applied stress changes from exponential for the soils with high Ip values (Çalışan samples) to polynomial for soils with low Ip values (Ingushetian silty clay).

3. The Russian criterion is seen to be between the Liquid and Plastic limit capacity boundaries. Thus, it is more conservative than the Gibbs criterion.

4. For a given soil type, once the soil state coefficient is determined, the following identification is straightforward. This may be used to regionally and lithologically classify collapsible soils.

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