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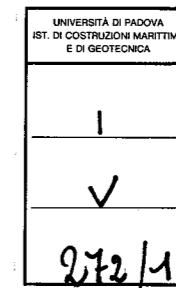
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viscosity of the permeant, dielectric constant is a significant factor affecting the permeability.

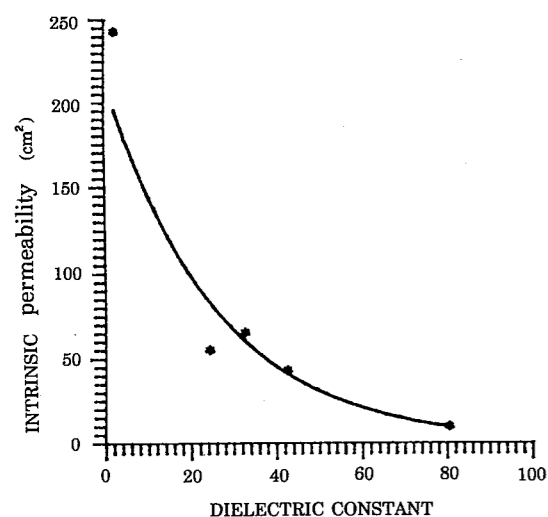


Fig.5 Relationship between Intrinsic Permeability and Dielectric Constant

CONCLUSIONS

A set of automatic triaxial flexible wall permeameters has been developed to study the effects of some organic fluids on the coefficient of permeability (K) of consolidated clays. Using this apparatus, the K values can be accurately measured, the effective stress or three dimensional pressures can be well controlled, and the errors due to side leakage can be avoided.

A consolidometer for preparing soil specimens from clay slurries has been designed, constructed and successfully operated. Using this method, the soil samples were found to be fully saturated, and their moisture or pollutant contents and void ratio were found to be nearly uniformly distributed. While preparing soil specimens, no corrosion has been found in the consolidometer.

In this study, a device, called "volume change buffer for pollutant", has also been designed, constructed and successfully operated. This device can be used to prevent the applied pollutants from getting into and damaging the sensitive sensors, and to insure that the volume change can be accurately measured and the pressure in the system can be fully transmitted without any loss.

Test results show that both dielectric constant and viscosity of the permeant are significant factors affecting the K values. In addition, the intrinsic permeability and the dielectric constant are highly related. Other test results and conclusions of this study will soon be published elsewhere.

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TENSILE STRENGTH TESTS ON COHESIVE COMPACTED SOILS

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SYNOPSIS. This paper deals with an experimental investigation carried out in order to evaluate some factors influencing tensile strength of cohesive soils. Split-tensile and Double-punch tests were performed and their results compared. Three base soils were used; a medium graded sand, a medium plasticity kaolin, and a high plasticity bentonite; the tested samples were created by mixing the base soils with different percentages to give soils of variable plasticity and by compaction according to Standard AASHTO procedure. Correlations among tensile strength, unconfined compressive strength, water content and plasticity index of compacted samples were investigated.

INTRODUCTION

The tensile strength of soil is generally considered small or equal to zero for engineering applications, because it is negligible with respect to compression strength or because simple satisfactory measuring equipment is lacking. There are many practical problems in which even low values of tensile strength may be very important for computational results, such as the cracking behaviour of earth dams or tension cracks in slope stability analysis.

The tensile strength of cohesive soils can be measured by applying either a uniaxial tensile force or a biaxial state of stress, failure being due to tensile stresses. The laboratory procedures used for determining the tensile strength of cohesive soils may be divided into the following main groups:

'direct tensile test': the sample is held at both ends and pulled apart. It is subjected to a uniaxial stress state and failure occurs when tensile stresses exceed the tensile strength of the soil sample. Specimen shape and preparation, type of tensile grip, and possible small bending stresses can significantly influence experimental results.

'bending test': a prismatic (or cylindrical) sample, simply supported at its ends, is loaded across its span, like a bending beam. Failure starts at the point of maximum tensile stress and tensile strength can be calculated by means of the theory of elasticity.

'hollow cylinder test': a hollow cylinder device is used for evaluation of the tensile strength and elastic properties of soil samples.

'triaxial test': the triaxial extension test can be performed to evaluate the tensile strength of soils. It consists of a controlled rate of strain test, with constant cell pressure ($\sigma'_1 = \sigma'_2 = \text{constant}$) and σ'_3 decreasing until failure.

'split-tensile or Brazilian test': a load applied along two opposite generatrices of a cylindrical sample produces tensile failure along the vertical diameter.

'double-punch or unconfined penetration test': a vertical load is applied to a cylindrical sample, using two rigid punches centered on both top and bottom surfaces, until failure.

This paper presents a series of experimental results carried out using the split-tensile and double-punch tests. The relationships among tensile strength, undrained shear strength, water content and plasticity index of compacted cohesive samples are investigated.

SPLIT-TENSILE TEST

The split-tensile test (ST test) was developed in the 1950s in order to evaluate the tensile strength of concrete: later it was also used in rock and soil mechanics. The ST test is carried out by placing a cylindrical sample horizontally between two rigid platens and loading along two opposite generatrices. For brittle materials the load produces a failure in tension along the diametral vertical plane. Frocht's equations, based on the theory of elasticity, are usable to determine tensile strength σ_t , as follows (fig.1):

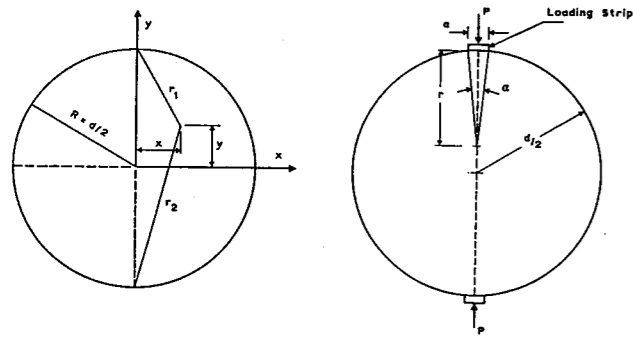


fig.1

$$\sigma_x = \frac{2P}{\pi t} \left(\frac{(R-Y) X^2}{r_1^4} + \frac{(R+Y) X^2}{r_2^4} - \frac{1}{d} \right) \dots (1)$$

$$\sigma_y = \frac{2P}{\pi t} \left(\frac{(R-Y)^3}{r_1^4} + \frac{(R+Y)^3}{r_2^4} - \frac{1}{d} \right) \dots (2)$$

$$\tau_{xy} = \frac{2P}{\pi t} \left(\frac{(R-Y)^2 X}{r_1^4} - \frac{(R+Y)^2 X}{r_2^4} \right) \dots (3)$$

where:

$\sigma_x, \sigma_y, \tau_{xy}$: stress components;
 P: applied load;
 t, d, R: height, diameter and radius of sample;
 r_1, r_2 : local coordinates (see fig.1).

Vertical stress σ_y along the X-axis is always a compression stress, ranging from a maximum value (+6P/ $\pi t d$) at the center to zero at the circumference; horizontal tensile stress σ_x ranges from a maximum tensile value (-2P/ $\pi t d$) at the center to zero at the circumference. Horizontal tensile stress σ_x along Y-axis has a constant tensile value of -2P/ $\pi t d$, while vertical stress σ_y ranges from +6P/ $\pi t d$ at the center to ∞ at the circumference. These high compressive stresses occurring along two loaded generatrices prevent failure in the central portion of the vertical diameter of the sample due to tensile stresses.

Applying a distributed load through a loading strip, the compressive stresses are greatly reduced and horizontal stress σ_x on the Y-axis changes its distribution, with compressive stresses close to the loading strips and tensile stresses at the center of the sample. σ_x assumes the following expression (fig.1):

$$\sigma_x = (1 - d(\alpha - \sin\alpha)/2a) 2P/\pi t d \dots (4)$$

If the tested soil has high compression strength, the sample may fail in tension in the central part of the loaded diameter.

DOUBLE-PUNCH TEST

The Double-punch test (DP test) was developed

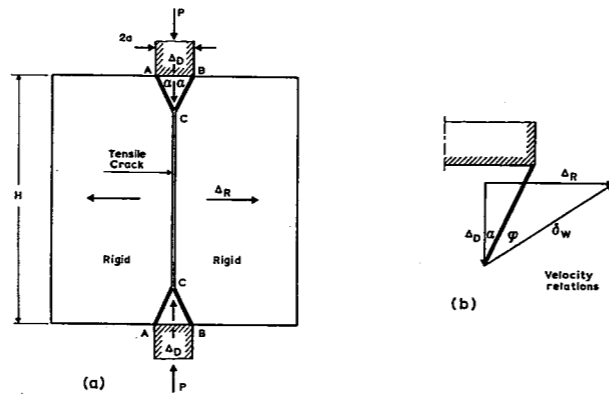


fig.2

at Lehigh University in order to evaluate the tensile strength of soils, rocks and non metallic construction materials. Failure in tension is produced by means of two rigid punches penetrating both the top and bottom surfaces of the sample, until it fails along radial vertical planes.

Tensile strength was theoretically determined by Chen and Drucker (1969), applying the upper bound theorem of the limit analysis method. Two fundamental assumptions were made:

1. sufficient local deformations in tension and compression must exist for application of the limit analysis method to the tested soils, idealized as perfectly plastic materials;
2. a modified Mohr-Coulomb failure surface in compression and a small but non-zero cutoff is postulated as a yield surface for the tested soil.

The theoretical failure mechanism consists of several radial tension cracks and two cone-shaped rupture surfaces, just beneath the punches. The conical surfaces move toward each other as rigid bodies, displacing the surrounding material sideways (fig.2a). δ_w is the relative velocity vector at each point along the cone surface and is showed in fig.2b with its radial and vertical components.

According to the upper bound theorem, it is possible to equate the external rate of work to the total rate of internal dissipation yields:

$$\frac{P}{\pi a^2} = \frac{1 - \sin\phi}{\sin\alpha \cos(\alpha+\phi)} \frac{\sigma_u}{2} + \tan(\alpha+\phi) \left(\frac{bH}{a^2} - \cot\alpha \right) \sigma_t \dots (5)$$

where:

P: upper bound of load;
 α : angle of cone;
 a: radius of punches;
 b, H: radius and height of sample;
 ϕ : undrained shear resistance angle;
 σ_u : unconfined compression strength.

The upper bound has a minimum value when α satisfies the following condition:

$$\delta P^u / \delta \alpha = 0 \dots (6)$$

By means of simple analytical considerations, equation (5) can be finally reduced to:

$$\frac{P}{\pi a^2} = \sigma_t \left(\frac{bH}{a^2} \tan(2\alpha + \phi) - 1 \right) \dots (7)$$

Using typical values of $\sigma_u, \phi, a, b,$ and H, Chen and Drucker proposed the following expression of σ_t :

$$\sigma_t = \frac{P}{\pi (kbH - a^2)} \dots (8)$$

with k=1.

TESTING SOILS

Three base soils were used for the laboratory investigation: a uniform graded sand, a medium plasticity kaolin and a high plasticity bentonite. The samples were created by mixing the base soils with different percentages to give soils of variable plasticity. The main index properties of the natural soils and the tested samples are summarized in tab.1.

The cylindrical samples, 103 mm wide and 115 mm high, were made by compacting the soil according to the Standard AASHTO procedure. Fifty-four samples were prepared, eighteen for each mixture. Eighteen ST, eighteen DP and eighteen unconfined compression tests (UC tests) were performed. For each group, six have been carried out on mixture 1, six on mixture 2 and six on mixture 3, varying the water content around the corresponding 'optimum water content' (OWC).

Compaction data are reported in fig.3. The OWC and 'maximum dry unit weight' (MDW), of each mixture are reported in tab.2. OWC increases and MDW decreases with increasing plasticity index I_p ; furthermore, the differences among dry unit weights are higher at moistures ranging between 15 and 25%, rather than between 25 and 30%.

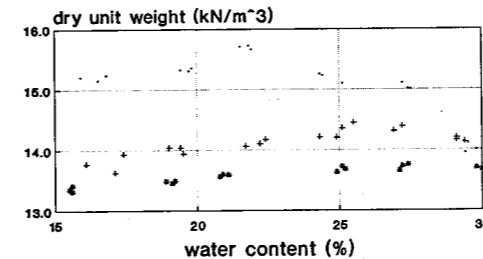


fig.3

tab.1

NATURAL SOILS	G	w _l (%)	I _p (%)
sand (s)	2.70	-	-
kaolin (k)	2.63	60	25
bentonite (b)	2.88	283	226

TEST MIXTURES			
1 (k 80%-s 20%)	2.64	48	15
2 (k 100%)	2.63	60	25
3 (k 80%-b 20%)	2.68	73	33

tab.2

MIXTURE	OWC (%)	MDW (kN/m ³)	σ_t (ST) (kPa)	σ_t (DP) (kPa)
1	21.8	15.45	48	42
2	26.5	14.42	54	41
3	27.5	13.75	67	51

Both kinds of test, ST and DP, were carried out in 'undrained conditions' using a rate of vertical displacement of 0.5 mm/min. It was observed that failure displacements are generally greater in DP tests than in ST tests.

Tensile strength and water content, for ST and DP tests and for each tested mixture, are plotted in fig.4. It is clear that tensile strength increases with water content up to a maximum corresponding to a 'critical' value (CWC) less than OWC; for values higher than CWC a sharp decrease in σ_t was noted in both tests.

Tensile strength values obtained with ST tests were always greater than those derived from DP tests. DP strengths were evaluated by equation (8) using coefficient K=1. Good agreement between ST and DP tests can be reached by considering K=0.8. However, keeping k=1 was preferred as it is not yet known whether the two testing procedures produce equivalent experimental results or whether one of them is preferable than the other. For all the mixtures investigated, a greater difference between σ_t derived from ST and DP tests was observed for water content less than CWC.

Fig.5 plots unconfined compression strength σ_u and water content for all mixtures. σ_u increases until w is less than OWC and then decreases. σ_u corresponding to w=OWC increases with the plasticity index.

Fig.6 plots tensile strengths σ_t against the plasticity index. Experimental data are compared to those proposed by Fang-Hirst(1973); the DP's values fit such a curve better than the ST ones. σ_t at OWC increases with increasing plasticity index.

Fig.7 plots ratios between σ_u and σ_t at OWC and the plasticity index. Data are compared to a curve proposed by Fang-Chen(1971). ST and DP strengths are arranged symmetrically with

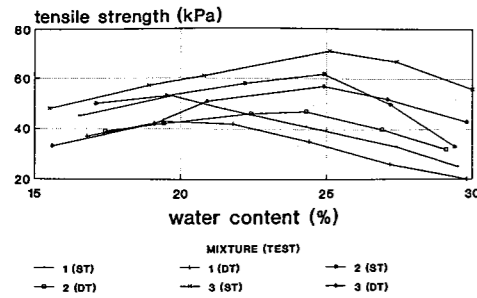


fig.4

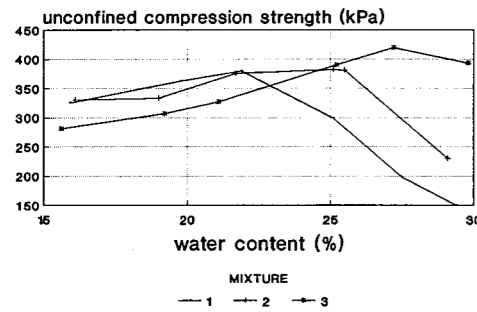


fig.5

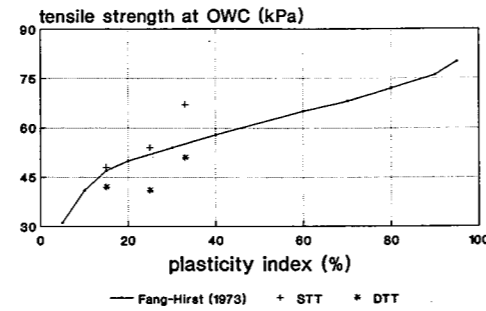


fig.6

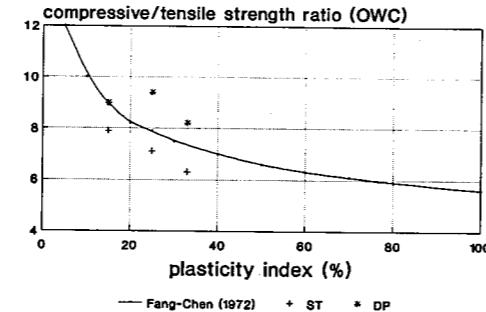


fig.7

respect to this curve. Furthermore, it is confirmed that, for $w_1 < 20+30\%$, ratio σ_u/σ_t is hardly influenced by either clay fraction or water content, while, for $w_1 > 30\%$, the water content becomes more important than the clay fraction.

CONCLUSIONS

The tensile strength increases as the plasticity index and liquid limit increase. It generally increases with water content w up to a critical value smaller than OWC; a sharp decrease in tensile strength then occurs.

Tensile strengths measured using ST and DP equipments were always different. Such a difference should be mainly due to:

- 1) DT test measures tensile strength on the weakest plane, whereas in ST test the soil sample fails across a predetermined plane;
- 2) the orientation of the compacted layers composing the sample is parallel to the load in ST test, and perpendicular in DP test: this fact influences the failure mechanisms in different manner.

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POST-CYCLIC MONOTONIC UNDRAINED BEHAVIOUR OF A MARINE CLAY

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SYNOPSIS A study of the influence of cyclic loading on the post cyclic undrained stress-strain and strength characteristics of a marine clay is presented. It is shown that the loss in undrained strength and stiffness as a consequence of cyclic loading are not uniquely related to the amplitude of strain during cyclic loading, as commonly assumed. Nor can they be explained in terms of pore pressures generated due to cyclic loading. A rational explanation of changes in post cyclic stress-strain and strength of clay when compared to those prior to cyclic loading is advanced in terms of hysteretic work absorbed by the clay during cyclic loading.

INTRODUCTION

An important consideration in the design of clay foundation for seismic or wave loading is the undrained response of clay during and after cyclic loading. Cyclic loading of clays causes, in general, a reduction in both stiffness and undrained strength on subsequent static loading. No systematic study has been carried out to assess this loss in stiffness, and there are conflicting conclusions as to the magnitude of strength reduction in studies reported in the literature.

This paper presents a study of the influence of cyclic loading on the post cyclic undrained stress-strain and strength characteristic of a marine clay. The influence of factors, such as, cyclic stress level, number of cycles, amplitude of maximum strain during cyclic loading and residual strain at the conclusion of cyclic loading is systematically investigated. In addition, the influence of initiating cyclic loading with the type of loading pulse (compression or extension) and the sense of residual strain in relation to the sense of strain during post cyclic static loading is studied.

EXPERIMENTATION

A local undisturbed marine clay (Cloverdale clay) was used in the study. It is a sensitive silty clay of soft consistency. The sensitivity (about 16) is attributed to surface infiltration following marine deposition and subsequent uplift. The clay was block sampled from an open excavation and all test samples were trimmed from the same horizon. This ensured maximum uniformity among various samples. The clay has average natural water content = 51%, liquid limit = 51% and P.I. = 27%.

All specimens were normally consolidated hydrostatically under an effective confining stress $\sigma'_c = 200$ kPa prior to cyclic loading. Cyclic loading was applied using a symmetric two-way sinusoidal deviator stress pulse at a frequency of 0.1 Hz. Under each cyclic stress level, cyclic loading was terminated when a prescribed magnitude of maximum axial strain was developed. This was followed by a period of pore pressure equalization prior to

carrying out post cyclic monotonic loading at an axial strain rate of about 1%/hour.

Static pre-cyclic undrained behaviour of the clay was determined in the normally as well as overconsolidated states using overconsolidation ratios ranging from 2 to 5. Both compression and extension loading modes were considered. These tests were intended to provide reference data for cyclic stress levels selected, and for assessing the equivalence between stress overconsolidation, and overconsolidation induced by cyclic loading.

STATIC AND CYCLIC LOADING BEHAVIOUR

Normally consolidated Cloverdale clay was found to exhibit normalized static undrained behaviour. The undrained strength ratio in compression $S_{uc}/\sigma'_c = 0.270$, and in extension, $S_{ue}/\sigma'_c = 0.24$. This undrained strength anisotropy caused a preferential development of larger axial strain amplitudes on the extension side during cyclic loading.

The development of maximum axial strain with number of cycles under various cyclic stress levels, defined as τ_{cyc}/S_{uc} , for the clay tested was similar to that reported for other clays (e.g. Andersen, 1975; Azzouz et al., 1989; Fischer et al., 1976; Takahashi and Hight, 1980). Both axial strain amplitude and residual strains increase with number of cycles at a given τ_{cyc}/S_{uc} , and for a given number of cycles increase with cyclic stress level.

Cyclic loading under identical cyclic stress level that initiated with commonly used compression pulse (C/E loading) resulted in extensional residual strains, whereas that initiated with extension pulse (E/C loading) gave rise to compressional residual strains (Fig. 1). This differing sense of residual strain has a dominant effect on post-cyclic clay behaviour, as discussed later.