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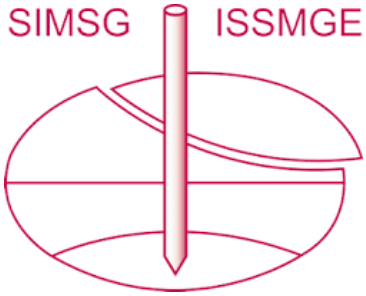

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Session 1

Stress-Deformation and Strength Characteristics

Relations Contrainte-Déformation et Caractéristiques de Résistance

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1. INTRODUCTION

The purpose of the General Report is to review and critique the 73 papers submitted to Session 1 (see Volume 1 of the Proceedings). This will be done in context of the State-of-the-Art (SOA) Report (see Volume 2 of the Proceedings) as follows:

Basic behavior of cohesionless soils

Basic behavior of cohesive soils

Computational and modelling capability

Evaluation of soil parameters, including both laboratory and in situ testing

Papers treating more than one topic are classified according to their major thrust and papers not falling within the above subject areas are covered under Miscellaneous topics. The papers are identified by underlining the names of the authors. Other references are listed at the end of the report.

The General Reporter received considerable assistance from Dr. Roger Foott, one of the Co-Reporters of the Session I SOA Report. In addition, three colleagues at the Massachusetts Institute of Technology provided helpful comments on many of the papers: Professor M.M. Baligh, Professor H.H. Einstein and especially Dr. S.M. Lacasse who also translated those papers written in French.

2. BASIC BEHAVIOR OF COHESIONLESS SOILS

Half of the six papers under this heading deal with the structure of sand. The very comprehensive and interesting paper by Oda and Koishikawa investigates anisotropy and its effect on drained strength and modulus. By measuring the orientation of the long axis of particles in three natural sands they conclude that the particles tended to be deposited with their long axes forming an acute angle with the presumed direction of water flow at the time of deposition. Using vibrated laboratory samples of 15 sands, the authors show that as the particles become longer and more platey (i.e. smaller axial ratio), there is an increased strength anisotropy. Tests on one sand also showed significantly greater strength anisotropy with a 10 fold decrease in confining stress. The paper presents results from model footing tests which yield a higher bearing capacity and modulus of subgrade reaction when the footing load is perpendicular to the bedding plane than when it is parallel to it. These tests also indicate increasingly significant anisotropy as the relative density increases. It should be noted that the authors of the paper relate anisotropy to particle orientation rather than to the orientation of interparticle normals as discussed in the SOA Report (Section 2.1.1). This different criterion presumably reflects the major contribution of particle orientation to anisotropic behavior. The indication that anisotropy is more significant at the higher relative densities is in direct conflict with the trends in Fig. 3 of the SOA Report. Whether this different behavior is due to method of sample preparation, particle shape, testing procedure and/or other factor remains unresolved.

Auvinet simulates by computer modelling the random deposition of both uniform (C_u = 1) and "non-uniform" (C_u = 2) spherical particles in a cubical space. Special attention is given to homogeneity, spatial distribution of contacts, number of contacts per particle, void ratio and anisotropy. The author presents very interesting results regarding the orientation of normals to interparticle contact planes that predict highly anisotropic structures for granular media, though the General Reporter found difficulty in understanding the physical meaning of some of the histograms. Auvinet further concludes that smaller particles existing within the granular matrix may take little part in the transmission of stresses, this having practical implications concerning the strength and compressibility of the soil mass.

Arthur and Assadi present extremely interesting data showing the influence of pre-existing rupture layers on the drained stress-strain-strength behavior of a dense sand. An ingenious reversible grouting technique employing a soap solution was developed to take samples which had already been sheared in a large direct shear box to just past the peak strength and to the maximum possible displacement. The specimens were then placed at varying inclinations in a cubical plane strain apparatus

having provisions for measuring internal strains using radiography. The presence of highly sheared pre-existing rupture layers caused ϕ' to decrease by about 5 to 15° compared to homogeneous samples, the amount depending upon the inclination of the rupture layer. Lightly sheared rupture surfaces had relatively little effect on strength, but caused very significant reductions in modulus. The authors present information on the divergence of the major principal axes of stress and strain increment resulting from the presence of rupture layers. The Leighton Buzzard sand used in this study exhibited marked inherent anisotropy, ϕ' decreasing by 5° as σ_{1f} rotated from the vertical to the horizontal direction. It is interesting to note that the direct shear ϕ' (assuming a horizontal failure surface) equalled the plane strain value for the corresponding direction of σ_{1f}. This apparently contradicts the discussion presented in Section 2.1.4 of the SOA Report based on the work by Rowe (1969).

Begemann et al. investigate the "liquefaction" potential of loose sands under static triaxial compression loading conditions in relationship to the "critical density" of the sand. Extensive data are reported from isotropically consolidated-drained and undrained tests (CID and CIU tests), both stress and strain controlled. The authors use the term liquefaction to mean a reduction in post peak undrained strength and also to signify substantial straining at essentially constant shear stress. Undrained stress and strain controlled shear tests at densities below the critical value develop large excess pore pressures and liquefy, as would be expected. The authors state that liquefaction can also occur in drained tests (both dry and saturated) if sheared under stress controlled conditions, though no stress-strain data illustrating this phenomenon are presented. An interesting theory to predict the effective stress path of CIU tests via stress-strain data obtained from CID tests is also proposed.

Using large scale simple shear and triaxial tests, Cullas studied the influence of gradation, d_{50} particle size, and relative density on the friction angle of granular materials of varying particle geometry. Unfortunately, little information is given regarding testing details and some important terms are not defined.

Kézdi and Horvath use a modified, automated triaxial compression cell to investigate the strains and volume changes in a partially saturated sand subject to various stress paths. The test results are presented as a function of the phase composition of the material and they report that the highest strengths are obtained at a water content lower than the Proctor optimum.

Papers by Nagaraj and Somashekar (Section 4 of this paper) Ohta and Hata, Rico et al. and Tanimoto and Noda (Section 6) also present data on the behavior of cohesionless soils.

3. BASIC BEHAVIOR OF COHESIVE SOILS

The 13 papers covered herein are divided into three major categories: soil structure, including physico-chemical phenomena and residual strength; influence of stress history and stress system; and creep effects.

Soil Structure

This fundamental topic was not covered in the SOA Report, partly due to space limitations but mainly because the subject is treated so well in the recent book by Mitchell (1976).

Changes in the microfabric (particle orientation and distribution) of resedimented clays as a function of consolidation and shear were studied by Matsuo and Kamon. Using the scanning electron microscope technique, the authors conclude that clay behavior over stress ranges of common interest is primarily controlled by deformation of particle aggregates, termed peds, rather than individual clay particles per se as often assumed. The paper proposes a classification system to describe the size of these peds and the corresponding pore size.

Krizek summarizes the principal results from a comprehensive investigation to relate, in a quantitative manner, the fabric (particle orientation) of Kaolin clay and its inherent anisotropic strength-deformation properties. Initially flocculated and dispersed samples were isotropically and K_o consolidated to different stresses and then subjected to a variety of fabric measurement techniques and varying types of shear tests (triaxial, lab vane, direct shear). The author discusses a number of interesting research topics ranging from creep behavior to development of a "micro-mechanistic" stress-strain model for clays. Since a great deal of material is covered in this one paper, detailed comprehension of the results will require study of the several referenced doctoral theses.

A series of oedometer tests on remolded kaolinite with different pore fluid characteristics (type of cation, pH, dielectric constant, etc.) is used by Genevois to investigate physico-chemical phenomena in pure clays. The pore fluid pH and dielectric constant have the most important effects on the location and slope of the e-log σ_c' curves. The results are discussed in terms of double-layer theory. However, as mentioned by the author, changes in pH also affect the edge charge on the particles. Since the behavior of kaolinite is primarily controlled by edge to face forces, rather than face to face (double-layer) forces, this would appear to complicate interpretation of the experimental data.

The dependence of the undrained strength of clays on the nature of the interparticle contacts and the degree of particle orientation is discussed by Sergeyev and Osipov. Perhaps due to problems with terminology (especially regarding exactly what types of

undrained shear tests were being performed) and since all data are apparently in terms of total stress, the General Reporter had difficulty in relating their work to that of others.

Stepkowska presents discussion and experimental data relating the effective stress and shear strength of bentonite to computed values of double-layer (osmotic) repulsive (P_R) and van der Waals attractive (P_A) forces with some rather surprising conclusions in the opinion of the General Reporter. For example, the shear strength of samples having a natural water content greater than the plastic limit is attributed to P_R being greater than P_A which appears to be contrary to the concepts of bonding based on measured activation energies (e.g., Mitchell, 1976).

The comprehensive and clearly presented paper by Kenney investigates the fundamentals governing the residual strength of soils. Extensive data on both natural clays and artificially prepared mixtures of massive minerals (e.g. quartz) and clay minerals (e.g. montmorillonite, hydrous mica, and kaolinite) clearly demonstrate the importance of mineralogical composition and physico-chemical factors such as type of exchangeable cation and pore fluid salt concentration. Thus empirical correlations between residual friction angle (ϕ_r') and index properties are generally only valid for deposits of similar composition and chemical state. Kenney points out that the porosity of massive minerals at their residual strength is usually 40 to 50%. Therefore, for soils in which the volume of the clay minerals and water (i.e. the "clay matrix") is less than 40 to 50% of the total volume, ϕ_r' is about equal to that of the massive mineral constituents. Kenney also found the residual strength to decrease in a fairly predictable manner for greater volumes of the clay matrix with most artificial and natural soils containing kaolinite and montmorillonite. But when hydrous mica was the dominant clay mineral, several unexplained anomalies exist. The chemical state had a pronounced effect on ϕ_r' of processed hydrous mica, but not with natural marine clays. Weathered marine clays had significantly higher ϕ_r' values (on the order of 10°) than unweathered clays of similar composition. Attempts to explain this fact by chemical treatment to simulate weathering proved unsuccessful. It should be noted that all data presented in the paper are tabulated for convenient reference. Also see Kanji and Wolle (Section 5) for a simplified procedure to measure ϕ_r'.

Stress History and Stress System

In the first of the three papers discussed under this heading, Josseaume and Pilot study undrained strength anisotropy and strain rate effects with three highly plastic (PI=60-67%) slightly overconsolidated clays (OCR=1.4 to 2.4). CK_oU triaxial compression and extension tests on samples consolidated to the net overburden stress (σ_{vo}') yielded $K_s=c_u(H)/c_u(V)$ ratios of 0.37, 0.48, and 0.62. The two lower numbers indicate a higher degree of c_u anisotropy for plastic clays than would be predicted based on the data plotted in Fig. 23 of the SOA Report. The values of K_o used in the tests (about 0.45 ± 0.05) are also less than expected, even for normally consolidated deposits [though consistent with those employed by Berre and Bjerrum (1973)] which might have contributed to the low K_s ratios. The authors also present data from "relaxation" tests on the three clays showing a 9 to 15% reduction in c_u per log cycle of strain rate (reference c_u at $\dot{\varepsilon}=0.6\%$ per hr). While these values fall within the $10\pm5\%$ range quoted in the SOA Report for CU triaxial compression tests, the authors' data extend to slower strain rates than generally used. The General Reporter would have expected less effect at very low strain rates, e.g. Fig. 10 of Berre and Bjerrum, (1973). This might be explained by the fact that the data came from relaxation tests rather than from tests sheared at different strain rates.

Balasubramaniam et al. investigate the undrained and drained stress-strain-strength behavior of two stiff Bangkok clays by means of CIU compression and extension tests and CID stress path compression tests (ranging from isotropic compression to loading at constant σ_3') for a range of consolidation stresses (σ_c'). Though values of maximum past pressure (σ_{vm}') for the soils were not reported, the results at different σ_c' are consistent with the expected behavior of clay with varying $OCR=\sigma_{vm}'/\sigma_c'$. The overall consistency in stress-strain-strength data is excellent and there was little difference in the Mohr-Coulomb effective stress strength parameters determined from the undrained TC and TE and the drained TC tests. The undrained strength anisotropy was relatively small, but this might be expected since CIU tests do not reflect the stress system induced component of anisotropy as defined in Section 2.2.2 of the SOA Report. The authors also used parameters obtained from the CID stress path tests to predict stress-strain behavior during standard CID compression tests and found excellent agreement with predictions based on the Revised Cam-Clay theory (Section 3.2.1 of the SOA Report).

Leon and Alberro performed CIU and CID TC and TE tests on undisturbed samples of the highly plastic Mexico City clay. A unique relationship was obtained between strength and void ratio at failure for all tests, an unusual finding since TE tests generally yield a lower e_f-log q_f line than TC tests because of the increase in intermediate principal stress. On the other hand, the ϕ' values (all very high for such a plastic clay) were found to vary with the type of test. Using cross-anisotropic elastic analyses of the drained tests, the authors conclude that the clay exhibits marked modulus anisotropy at low consolidation stresses. Hvorslev parameters are given

for the soil, but these are subject to considerable uncertainty since they are based almost entirely on data from normally consolidated tests. Normalization of the stress-strain curves presented in the paper would have facilitated interpretation of the influence of consolidation stress on test results.

Overall, few of the Session 1 papers present new information concerning the influence of stress history and stress system on clay behavior, more emphasis being placed on development of models which consider these effects, as discussed in Section 4.

Creep Behavior

Lacerda treats the topic of undrained stress relaxation at constant strain. In an excellent earlier paper on the subject, Lacerda and Houston (1973) showed that stress decays linearly with log time at a rate which can be predicted with reasonable accuracy from the Singh-Mitchell three parameter creep equation (Eq. 2.7, Section 2.2.6 of the SOA Report). They further demonstrated that the "effective" time at which stress relaxation starts to occur was uniquely related to the prior strain rate, provided that $D=\Delta q/\Delta q_f$ was constant (most data were for D=unity, i.e. for samples carried to failure before initiation of stress relaxation). In the present paper, Lacerda shows greatly reduced rates of stress relaxation if the sample is first partially unloaded after reaching failure, as would be expected. He also maintains that the stress acting on the "unloaded" sample will eventually coincide with the "conventional" test (i.e. when stress relaxation starts from D=unity), but little data are presented to support this hypothesis.

Ter-Stepanian discusses the drained (inferred by the General Reporter since not specifically stated by the author) creep behavior of clays as being characterized by the two phases described by Vyalov et al. In the first, termed "mobilization" when the strain rate is decreasing with time, deformation follows rate process theory (Section 2.2.6 of SOA Report). During this deterministic process, the strength of the interparticle bonds increases with time. In the second, termed "rupture" when the strain rate starts to increase with time, changes in soil structure follow a stochastic process. With undrained creep, this second phase would be expected to eventually lead to failure, i.e. creep rupture (Fig. 42 of the SOA Report). But during drained shear, the author presents data showing that the increased strain rate continues only to a certain "stabilization limit", whereafter the strain rate decreases. Ter-Stepanian notes that this important finding agrees with the long term creep data reported by Bishop and Lovenbury (1969). For the overconsolidated plastic clay tested, both creep phases were noted for all values of shear stress level between the "creep limit" ($\tau/\sigma' \simeq 0.15$) and that eventually leading to failure ($\tau/\sigma' \simeq 0.55$).

Time dependent displacements of excavated slopes in an overconsolidated jointed clay prompted Esu and Grisolia to study the creep characteristics of this soil via stress controlled CD direct shear tests run on intact samples sheared parallel to the natural layering. At stress levels (τ/τ_f) between 0.4 and 0.9, the horizontal displacement rate ($\dot{\delta}$) reportedly decreased to an approximately constant value (over a 2 month period) about 20 days after loading, this value increasing with stress level. The authors might consider plotting log $\dot{\delta}$ vs. log t or log δ to better compare their results to those reported by Ter-Stepanian and Bishop and Lovenbury (1969). Data on samples previously sheared to the "residual" strength exhibited similar behavior, which would appear to contradict the common belief that the residual strength is not particularly sensitive to strain rate.

Yudhbir and Mathur present needed data on the stress path dependency of drained creep from results of stress controlled triaxial compression tests run on an anisotropically consolidated resedimented plastic marine clay. Below a "yield value", the axial creep rate (strain per log cycle of time) increased linearly with stress ratio (q/p') for stress paths that would eventually lead to failure (as might be expected), whereas the volumetric creep rate was independent of the increase in q/p' for a given initial set of consolidation stresses. The authors propose a creep model using parameters obtained from contant p' and constant q creep tests that apparently worked well, though the General Reporter encountered difficulty in understanding exactly how it should be applied.

Josseaume and Pilot present data on undrained creep and several of the papers discussed in Section 4 develop creep models, e.g. Sekiguchi and Vyalov et al.

4. COMPUTATIONAL AND MODELLING CAPABILITY

The 19 papers considered under this heading are divided into three categories: those which focus on yield criteria and failure laws; non-viscous stress-strain models; and time-deformation models.

Yield Criteria and Failure Laws

Tavenas and Leroueil develop a model which explains the yielding behavior of the sensitive Champlain clays in a most lucid and interesting manner. From results of CIU, CID and constant $K(=\sigma_h'/\sigma_v')$ triaxial tests and oedometer tests, they show that the yield locus (point of marked change in stress vs. strain curve) plotted in terms of $0.5(\sigma_1'-\sigma_3')$ vs. $0.5(\sigma_1'+\sigma_3')$ has an elliptical shape centered about the normally consolidated K_o line rather than the isotropic stress axis as assumed in the Cam-Clay model. The locus intersects the K_o line at a value of σ_1' equal to the maximum past pressure (σ_{vm}') developed by geological

precompression and/or aging (i.e. secondary compression). Consequently, yielding during isotropic consolidation occurs at a stress significantly less than σ'_{vm}. The normally consolidated K_o is taken as $0.9(1-\sin\phi')$ and thus the yield locus is solely determined by ϕ' and σ'_{vm} (at least conceptually since no equations are given to define its exact location). The authors use this model to explain the influence of creep, both drained and undrained, on stress-strain-strength behavior. The yield locus expands with aging (i.e. increased apparent σ'_{vm} due to secondary compression) and contracts with decreasing strain rate during undrained and drained loading (e.g. slower rates of loading cause a decrease in the apparent σ'_{vm} of a clay). Fig. 13 of the paper and the related discussion on this topic are particularly worthy of study. The authors also comment on the implications of their model regarding present design practice, placing emphasis on interpreting strength data in terms of the effective stress paths resulting from varying types of laboratory and in situ shear conditions. In addition, the authors attribute the measured σ'_{vm} of the Champlain clay to aging and/or geologic erosion rather than to natural cementation (e.g. Sangrey, 1972). The General Reporter questions this hypothesis in view of the very high liquidity index of these clays in comparison to their c_u and σ'_{vm} values. Nevertheless, the paper is highly recommended for those interested in obtaining insight into many of the important factors affecting the deformation characteristics of clay.

Lade and Musante performed isotropically consolidated-undrained (CIU) "true triaxial" tests to study the effect of the intermediate principal stress (σ'_2) on the effective stress failure envelope of remolded Grundite clay. The peak friction angles with $\sigma'_2>\sigma'_3$ were higher than those obtained in triaxial compression and agreed reasonably well with the failure criterion proposed by Lade and Duncan (1975) for sands, namely I_1^3/I_3 = constant (determined from TC tests) where I_1 and I_3 are the first and third stress invariants respectively. Unfortunately, the paper does not give the actual variations in ϕ'. The authors then discuss plastic potential surfaces, present data showing the directions of strain increment vectors at failure, and conclude that the basic elastoplastic model developed by Lade and Duncan (1975) for sands from concepts of plasticity theory can be similarly applied to normally consolidated remolded clay. However, since this theory assumes isotropic material properties, the model proposed by Prevost and Hoeg would appear better suited to describe the undrained stress-strain behavior of natural clays.

Nagaraj and Somashekar modify the Extended von Mises failure criterion to give equal values of ϕ' in triaxial compression and extension and use the resultant equation to predict the influence of the intermediate principal stress (σ'_2) on strength. A "true" triaxial apparatus is described and strength data from drained tests on a coarse sand and kaolinite presented as a function of $b=(\sigma'_2-\sigma'_3)/(\sigma'_1-\sigma'_3)$. The measured ϕ' values for both soils increased until b reached 0.5 to 0.6, and then decreased as b approached unity, as predicted by their failure criterion. Note, however, that ϕ' in triaxial extension is usually greater than the compression value, at least for sands (e.g. Fig. 13 of the SOA Report), and thus the failure criterion proposed by Lade and Musante should prove superior for most cohesionless materials

Akai et al. investigate the yielding and creep behavior of a soft sedimentary rock (porous tuff) via strain controlled CU and stress controlled CD triaxial compression tests. The "break" in log-log plots of deviatoric stress vs. strain defines the point of plastic yielding and a plastic potential surface is developed using the associated flow rule from classical plasticity theory. The normality condition was approximately satisfied for this material that expanded during plastic deformation. The authors then develop "dynamic" yield loci from drained creep tests to describe the influence of strain rate on deformation behavior at stress levels exceeding the "static" yield locus, considered as the long term strength of the material. The results are to be applied to deformation and stability problems related to tunnel and bridge construction involving soft rocks.

Hashiguchi and Ueno develop a new, complex failure criterion for normally and overconsolidated clay. Hvorslev's extension of the Mohr-Coulomb equation is redefined and modified using concepts based on plasticity theory and the existence of a critical state. Published data on the Weald clay are used to evaluate predicted and measured results. The General Reporter could not understand the basic reasoning underlying development of the proposed criterion and thus cannot comment on its general applicability.

Davidson investigated the "yield surface" of Grundite clay aged to produce a quasi-preconsolidation pressure. Drained triaxial compression tests were run along different stress paths (isotropic compression, loading at constant σ'_3, $\Delta\sigma'_{oct}$ and $\Delta\tau_{oct}/\Delta\sigma'_{oct}$, etc.) on samples isotropically consolidated at three pressures (σ'_c) for 23 days. Yielding was defined from the "break" in "straight line" plots of $\Delta\sigma'_{oct}$ vs. volumetric strain and $\Delta\tau_{oct}$ vs. shear strain. It is difficult to evaluate this approach since stress-strain data from only one test are presented. The author then develops a three-dimensional effective stress ellipsoidal shaped yield surface in terms of σ'_c, $\Delta\sigma'_{oct}$ and $\Delta\tau_{oct}$. The model has been incorporated into a finite element program that reportedly gives results in agreement with field observations, but few details and no references are given.

Non-Viscous Stress-Strain Models

Four papers dealing with "simplified" stress-strain laws are considered first, then those which propose "generalized" stress-strain models.

Schultze and Teusen develop equations to predict consolidation settlement for axi-symmetrical loading of normally consolidated soil using the hyperbolic stress-strain relationship of Duncan and Chang (1970). Superposition is assumed to apply such that data from simple tests (e.g. standard CID triaxial compression) can be used to predict stress-strain response for varying stress paths. Experimental data for a silt are given, but little information is presented regarding the validity of superposition nor are predicted settlements using the proposed method compared to other procedures or field data.

Shackel and Makiuchi use cyclic "true triaxial" tests to investigate the influence of variations in the number of cycles and the intermediate principal stress on the quasi-elastic strain behavior (i.e. recoverable strains upon unloading) of a compacted sand-clay. A linear, isotropic deformation law is used to model the essentially isotropic response of the soil.

Starzewski and Thomas found the axial stress vs. axial strain and axial strain vs. volumetric strain relationships obtained from CD stress path triaxial compression tests on horizontal and vertical samples of heavily overconsolidated (K_o>3) Lias clay to be linear up to axial strains of about 1%. The material was highly anisotropic, with a horizontal modulus several times larger than the vertical. The test results were interpreted to give cross-anisotropic elastic parameters which were used with good to reasonable success to predict the behavior of isotropic consolidation and constant stress-increment-ratio drained tests. Attempts to predict incremental loading test behavior give reasonable to poor results. This thorough and detailed paper should be of interest to researchers working in the area of elastic modelling of soil behavior.

Korhonen develops mathematical relationships to model effective stress paths observed in CIU triaxial compression tests on an overconsolidated plastic clay and uses the well known hyperbolic equation to describe stress-strain behavior.

In one of the most clearly presented papers concerning "generalized" models, Prevost and Hoeg propose a general analytical model to describe the anisotropic, elastoplastic, stress path dependent effective stress-strain-strength behavior of saturated clays under undrained loading conditions. The initial positions and sizes of the yield surfaces before undrained shear are defined by the stress history of the soil. The yield surface at the peak undrained strength can be determined either from K_o consolidated-undrained triaxial compression and extension tests or from a direct simple shear (DSS) test with measurement of the stress in the reinforced, cylindrical rubber membrane. Data from TC and TE tests on the Bangkok (OCR=1) and Drammen (OCR=1 and 4) clays are used with the model to predict stress-strain behavior in direct simple shear tests, with excellent agreement being obtained. A method is also presented to determine σ'_h/σ'_v in DSS tests at failure. The model, suited for use in finite element analyses, appears most promising for practical applications since the parameters can be determined from fairly simple tests and it accounts for stress system induced anisotropy.

Herrera et al. present a theory of stress-strain behavior for isotropic "wet" clays based on using an "equivalent preconsolidation pressure" as an internal state variable. The Cam-Clay model is discussed in light of the theory where it is "shown" that the original hypothesis regarding elastic shear distortions is inconsistent with the assumed volumetric behavior and actual experimental data. In the new theory, elastic distortional deformations are controlled by the magnitude of the equivalent preconsolidation pressure rather than the mean effective stress.

Juarez-Badillo's highly mathematical paper develops a generalized non-linear theory to describe the deviatoric stress-strain behavior of normally consolidated clays. A relationship is developed among the peak friction angle, K_o, and coefficients of volumetric compressibility and shear deformability. Using experimental data on the Weald clay, the theory is applied to drained triaxial compression and extension tests and one-dimensional compression behavior. The General Reporter cannot judge the technical merits of the model which appears primarily of interest to researchers.

Sobotka et al. develop a model to predict the stress-strain behavior of soils considering anisotropic material properties and the effects of dilatancy with a yield criterion that depends on the "sign" of the applied shear stress. The formulation is very complex and could not be understood by the General Reporter. Experimental data from CD direct shear tests on inclined samples are presented to study the influence of the sign of the shear stress. The paper also contains results of constant volume and constant normal stress tests on loose and dense sand.

In the paper by Jacobsen, oedometer, CIU and CID stress path triaxial compression tests are used to obtain four empirical state functions to describe the dependence of volumetric and distortional strains on the isotropic and deviatoric components of stress for an overconsolidated moraine clay.

Noting that extensive testing is required to determine the constitutive relationships in many elastoplastic models, Battelino and Majes suggest that a more direct interpretation of experimental data may suffice

for many practical applications. Using results from triaxial compression tests interpreted in terms of octahedral values of stresses and strains, the authors develop a "hybrid hypoelastic" model for use with the finite element method to predict the undrained and drained response of a marsh clay to an embankment loading. Among other simplifications used in the analyses, it should be noted that shear moduli were assumed equal for undrained and drained conditions.

As previously noted, the elastoplastic model proposed by Prevost and Hoeg appears best suited to describe the anisotropic undrained deformation-effective stress behavior of soft clay deposits.

Time Deformation Models

Surprisingly none of the Session 1 papers consider creep effects during consolidation though this topic is one of considerable controversy and, perhaps, practical significance. However, two deal with the important subject of creep per se.

Sekiguchi presents constitutive relations to describe the visco-elastoplastic behavior of normally consolidated clays using five parameters, of which at least three can be readily evaluated from conventional tests. The General Reporter can not comment on the theoretical equations, but is impressed by their apparent ability to model the undrained viscous behavior of clays. This includes the effect of strain rate on stress-strain-effective stress behavior, stress relaxation versus time and undrained creep, including creep rupture wherein the "upper yield strength" is given by an equation similar to that used in the Cam-Clay model. The computed results, based on assumed material properties, are presented in a concise and clear manner. Though the model contains several simplifying assumptions, it appears to offer considerable potential for interpreting the rheologic behavior of clays and for use with the finite element method to predict field performance.

The first portion of the paper by Vyalov et al. extends previous research by Vyalov and his co-workers in applying the principles of a kinetic theory of deformation to explain the creep behavior and long term strength of cohesive soils. Based on a study of changes in the microstructure of soil during deformation and by considering changes in the activation energy with time, the authors develop a generalized creep equation that can be used to model different types of creep behavior (strain rate increasing, constant or decreasing with time). They postulate two major processes occurring in the microstructure of soil, one causing a strengthening and the other causing a weakening of the structural bonds with time. Parameters are presented to describe these two phenomena, though it is not clear to the General Reporter exactly how they are determined experimentally. The second portion of the paper discusses the application of the theory to solve practical problems. This apparently entails evaluation of three components of strength: the "true" angle of friction, an "irreversible" cohesion (e.g. cementation) and a "reversible" cohesion; and how these components vary with time and amount of shear deformation. Interestingly their equation can model changes in creep rate with time that agree with the experimental data presented by Ter-Stepanian. This work may well prove to be an important contribution to creep modelling, at least regarding the concepts presented in the first part of the paper.

Hanrahan treats soil as a "dimensional-dependent" material in studying the one-dimensional consolidation behavior of "large-strain" soft, saturated soils. Having postulated separate shear and volumetric components of strain, the author concludes that small scale laboratory tests cannot be used to reliably predict field behavior because of different scaling factors for the two components of strain, i.e. it is not possible to relate total strain to effective stress only without reference to time. The General Reporter concurs with Green (1971) that the validity and practicality of this approach to analyzing soil behavior appear to be uncertain.

5. EVALUATION OF SOIL PARAMETERS

Of the 20 papers included under this category, about half deal with laboratory equipment and testing procedures. The others concern in situ testing and comparison of results obtained from in situ and laboratory tests and from field performance data.

Laboratory Testing

Six papers deal with new devices and testing procedures, two show the effects of macrofabric on test results, and one concerns centrifugal model testing.

The Cambridge Simple Shear Apparatus and the data acquisition system used by Liege University to operate the device are described in the paper by Fagnoul et al., along with results from several drained tests on a uniform river sand.

Nascimento describes a new approach for measuring the interparticle friction angle (ϕ_μ) of granular materials using a goniometer. In contrast to other devices, it can test two natural rough surfaces in sliding. Possible errors associated with the test are discussed in detail and data presented showing the large scatter in ϕ_μ values for granular materials obtained from testing 5 particles in pairs. While on this topic, reference should be made to Skinner (1969) who clearly demonstrated that there is no direct relationship between ϕ_μ and the frictional behavior of cohesionless soils because of the influence of particle shape and hence interparticle rolling as opposed to simple sliding.

Kanji and Wolle propose a very simple procedure for measuring the residual

strength of clay soils by shearing them along an interface with a hard polished surface in a standard direct shear device. The sample reaches its minimum strength at small displacements which obviates the need to use multiple reversals of shear in the test to achieve the residual state. The average roughness of the hard surface should be on the order of the maximum size of the clay particles and electron microscope photographs are presented showing an almost completely parallel orientation of clay particles at the sheared contact surface. Good agreement is noted between the ϕ_r' vs. PI relationship from this procedure and relationships reported by others. This suggests considerable merit for this simple test, though direct comparisons for a variety of clay types would appear to be in order.

The paper by Chang et al. presents a new method for estimating the in situ K_o of normally and lightly overconsolidated clays. It consists of first isotropically consolidating a sample in a standard triaxial cell to a stress slightly less than the in situ effective horizontal stress and then holding the cell pressure constant while increasing the vertical stress. Total volumetric and axial strains are monitored and when they are equal, the corresponding value of $\sigma_{hc}'/\sigma_{vc}'$ is equated to K_o. Data on two natural and two sedimented clays show the resultant K_o values to be larger than those obtained by allowing no lateral strain throughout consolidation to the in situ net overburden stress, the "traditional" method which the authors believe will underestimate the in situ K_o because of sample disturbance effects. On the other hand, the K_o values agreed very well with those determined by the Poulos and Davis (1972) method, which the authors consider reliable when applied to normally consolidated clays. Thus the new method apparently has promise and might well be tried by others, though further explanation of the rationale underlying the procedure and typical stress-strain data during such tests would be welcomed.

The criteria recommended by Bishop and Henkel (1962) for ensuring adequate pore pressure dissipation in drained triaxial tests are considered inadequate by Thurairajah and Balasubramaniam, who develop a new theory based on the incremental pore pressures generated during testing. The expression developed for determining suitable strain rates requires an estimate of the generated pore pressure vs. strain relationship, which is assumed to be linear and thus not applicable to overconsolidated clays. No experimental data are presented and the theory does not directly account for the fact that filter strips generally do not provide full radial drainage.

Parez and Bachelier propose a method for estimating the in situ maximum past pressure of cohesive soils using oedometer and triaxial tests based on consideration of "elastic potential energy" under "uni-axial and spherical" states of stress. Their approach is unusual and difficult to understand.

By means of laboratory tests on stiff fissured glacial tills and soft laminated alluvial silty clays located in Scotland, McGown et al. investigate the relationship between the nature and distribution of the fissures/laminations and the measured parameter values. Data show that the drained and undrained strength of the glacial till are strongly influenced by the spacing and orientation of the fissures in relation to the specimen size and by the nature of the fissure coatings. The compressibility and permeability characteristics of the alluvial clay were significantly affected by the number of laminations included in the samples. The authors give practical guidance regarding test procedures required to obtain laboratory results representative of in situ behavior.

Rizkallah presents triaxial and direct shear data showing the substantial influence of fissures in overconsolidated Hannover clay on the Mohr-Coulumb envelope. He presents electron microscope photographs showing parallel alignment of particles in defined shear zones within tested samples, with random orientation elsewhere in the specimen. He comments on the difficulty of establishing empirical correction factors to use with laboratory test data to give economical design and on the need to consider each stiff fissured clay deposit individually since variations in geological and prestress history cause different behavioral trends.

The well presented paper by Pincent et al. describes in detail the testing procedures and results of centrifugal testing to model the behavior of a large instrumented excavation in soft organic clay that was rapidly cut to produce slope failures. Prediction of pore pressure behavior with the 1:100 scale model was considered poor, perhaps due to the simplified procedure used to simulate the excavation process (i.e. the test was stopped in order to make the cut). Vertical deformations were generally of the correct order of magnitude and largest in the most critical regions, but the model did not predict the observed failures. The authors suggest that organic debris in the soil may have prevented failure in the model. It should be noted, however, that behavior must be governed by strains, not displacements, when attempting to predict prototype performance by centrifugal testing.

In Situ Testing and Comparative Studies

Seven of the 11 papers in this category concentrate on evaluation of undrained strength-deformation parameters, while the others mostly deal with predictions of long term settlements. We shall first discuss four papers on field vane testing and then three on more sophisticated forms of in situ testing for undrained parameters.

Donald et al. investigate in considerable detail different procedures for evaluating

the results of field and lab vane tests. The data clearly demonstrate that the vane test can not be used to give reliable estimates of undrained strength anisotropy and the authors conclude that c_u anisotropy is better evaluated using other techniques. They also present initial results from an ongoing finite element study of the stress conditions during a vane test and note that the research should ideally be extended to consider stress variations around the circumference of the sheared cylinder. However, they question whether such work will result in an increase in the usefulness of vane results, presumably agreeing with the SOA conclusion (Section 4.2.4) that the FVT is best viewed as a semi-empirical strength index test.

Costa Filho et al. summarize undrained strength data on a highly plastic (PI=140%) soft, sedimentary, lightly overconsolidated clay (OCR=1.8 attributed to aging) in a well presented paper. A program of field vane tests with varying h/d ratios (0.5, 1 and 2) and UU triaxial compression tests on vertical, 45^0 and horizontal samples indicated little strength anisotropy, which is consistent with the trends described in Section 2.2.2 of the SOA Report. It should be noted, however, that UU tests will, at best, only indicate the inherent component of anisotropy while use of FV tests with different shapes is not considered a reliable procedure for measuring anisotropy (Section 4.2.4). The FV strength anisotropy, calculated via the method proposed by Aas (1965), was one-third higher for an assumed triangular distribution of shear stress over the end surfaces. The authors plan to reinterpret the data using the Wiesel (1973) method which considers variation in the peak values of S_H and S_V with the angle of vane rotation. Measurement of the preshear effective stress ($\bar{\sigma}_s$) in the UU tests yielded $c_u/\bar{\sigma}_s$ ratios showing more scatter than c_u/σ'_{vo} (this should have been expected since c_u is not linearly related to $\bar{\sigma}_s$). Values of E_u/c_u at a 50% stress level of about 100 from both UU tests and normally consolidated CIU triaxial compression tests are consistent with the low numbers presented in Fig. 19 of the SOA Report for direct simple shear tests on highly plastic clays.

Bozozuk analyzes three bearing capacity failures of tower silos using Skempton's (1951) equation with c_u values averaged over a depth equal to two-thirds of the silo diameter. The three sites all had stiff desiccated clay crusts overlying medium to soft clays of high liquidity index (one varved deposit and two Champlain clays). Strengths were determined from field vane, unconfined compression and either CIU or CAU triaxial compression tests reconsolidated to or somewhat below the net overburden stress. The author concludes that bearing capacities computed from average FV strengths were reasonably accurate (the PI of the clays was such that Bjerrum's correction factor is 0.85 or greater). c_u values from CU tests were less than FV strengths in the upper crust at all three sites and also at depth for the varved deposit, this difference being attributed to sample disturbance (the General Reporter has some doubts regarding this explanation). At the Champlain clay sites, CU tests yielded higher peak strengths in the underlying softer material than FV tests and their use throughout overpredicted the factor of safety. Bozozuk concludes that CU peak strengths should not be used to evaluate stability since they cannot be simultaneously mobilized along the entire failure surface, whereas FV data automatically account for this due to progressive action inherent to the test. While this phenomenon is no doubt important, differences in FV and CU triaxial compression strengths are also significantly influenced by other factors such as c_u anisotropy. Finally, it should be noted that all three sites had very substantial variations in c_u with depth within the failure zone. Thus, while the average c_u values used in this study can be compared for design purposes, it does not necessarily follow that the FV strength profiles are an accurate representation of the variation in the in situ strength with depth.

Law et al. report undrained strength data at three sites (highly sensitive Champlain clay) beneath embankment fills wherein significant consolidation settlements occurred due to loading in excess of the maximum past pressure over a portion of the foundation soil. No increase in the field vane values of c_u was measured, whereas CIU and CAU triaxial compression tests showed a substantial strength increase. The authors suggest that the FV tests did not properly reflect a strength gain because the test measures the strength on vertical surfaces and changes in effective consolidation stress on this surface were very small. The data presented clearly demonstrate the shortcomings of the FV test in evaluating strength increases with certain soil types and loading conditions. However, it is difficult to assess the actual strength increase that would be expected in situ since the quoted changes in c_u obtained from the CU test program were not related to the computed changes in consolidation stress.

The details of the self boring lateral penetrometer mentioned in Section 4.2.8 of the SOA Report are described in the paper by Baguelin et al. . The device (called PENLAT) uses a 10 cm^2 cylindrical flat-end probe and the resultant stress-deformation curve is interpreted to give a constrained modulus and a penetration limit resistance. Data from three sites compare PENLAT results with PAFSOR and Menard pressuremeter modulus values and Dutch cone data. As with most in situ tests, simplifying assumptions and empiricisms are involved in deriving soil parameters from this device and further experience will be needed to establish its relative merits compared to other types of in situ testing.

A comprehensive series of screw plate tests (Janbu and Senneset, 1973) were performed

by Schwab and Broms to investigate the in situ undrained strength and time-deformation behavior of three soft cohesive soils. The program included both constant load vs. settlement tests at different stress levels and incremental stress tests with different time intervals between the load increments. By using a procedure similar to that used by Singh and Mitchell in their rate process theory for undrained creep (Section 2.2.6 of the SOA Report), the authors develop mathematical relationships between stress level, settlement and time. Generally good agreement is found between calculated and measured results and the trends are in accord with expected undrained creep behavior. The approach appears to have merit for evaluating the influence of time with short duration screw plate tests (i.e. for basically undrained conditions) but should not, in the opinion of the General Reporter, be used to predict long term performance wherein the effects of drainage become increasingly important.

Windle and Wroth present comprehensive c_u, E_u and K_o data from Cambridge self-boring pressuremeter tests on the Gault and London stiff clays and compare the results to those obtained from plate load, Dutch cone and laboratory UU triaxial compression tests. Peak values of c_u were computed from the slope of the pressure-strain curves (Eq. 4.8, Section 4.2.6 of the SOA Report) and average values from the pressure-log strain data. Increasing the strain rate by a factor of 3 caused a 30% increase in peak strengths but little change in average strengths and modulus with Gault clay. It is interesting that the fast tests exhibited considerable strain softening whereas the slow tests did not. For the London clay at Hendon, peak c_u values were about 20% larger than the average, which in turn exceeded strengths obtained from large diameter plate load tests by about 30%. SBPT E_u data at this site agreed quite well with those derived from plate load tests. There was considerable scatter in lateral stress measurements, attributed in part to variations in the drilling technique, but average values of K_o were considered reasonable. In this well presented paper, the authors have clearly established some of the principal factors affecting SBPT data in stiff clays and their results should be studied by all interested in pressuremeter testing.

Garga and Costa used in situ 30 cm diameter plate load and screw plate tests and laboratory unconfined compression (cut at varying inclination) and oedometer (varying size) tests to investigate the deformation characteristics of a partially saturated residual ML soil weathered from gneiss. The vertical compressibility measured via plate load tests at depths up to 8 m was several times less than obtained from screw plate tests and laboratory data, this significant difference being attributed to disturbance and sample size effects (the authors note that field settlements of residual soils are much less than predicted from conventional oedometer tests). Vertical and horizontal plate load tests and the UC tests showed larger compressibility in the horizontal direction in the top 6 m, but not at greater depths where the soil was less plastic (perhaps due to less weathering, though not commented on by the authors). The paper briefly summarizes the effects of load increment ratio and duration on oedometer test results and reports an incremental K_o ($=\Delta\sigma'_h/\Delta\sigma'_v$) of about 0.6 for the soil.

Jones compares the deformation vs. log time plots for silty sand, clayey sand and silty clay soils obtained from consolidometer tests and by placing a "small" static load on a cone penetrometer inserted in the soil. He found the two sets of plots to be qualitatively similar and suggests that empirical techniques could be developed to allow preliminary estimation of consolidation characteristics based on "constant stress penetrometer" test data.

The paper by Schultze and Biedermann presents empirical correlations among results from different types of pressuremeter and dynamic and static penetration tests and laboratory oedometer tests based on extensive data collected in several countries on soils ranging from sand to clay. These include relationsnips between the modulus determined from Menard and Leischner pressuremeter tests and: dynamic and standard penetration N values; Begemann cone resistance; and modulus values from oedometer tests interpreted according to Janbu. Empirical equations are presented, which include the effect of overburden stress, and discussed in terms of correlation coefficients that generally ranged between 0.75 and 0.85. However, no consideration was given to soil type and whether the test data would correspond to undrained or drained conditions. These factors must affect scatter in the correlations and hence their usefulness.

Walker and Morgan review field performance of a firm to stiff silty clay deposit subject to embankment loadings and excavations. They report, for example, measured rates of consolidation 200 times faster than predicted from laboratory consolidation tests and conclude that "an uncritical application of conventional theory together with standard laboratory testing procedures can lead to gross errors in the prediction of field performance" with this deposit. This material with its silt and sand lenses does indeed appear to present design problems. Nevertheless it seems reasonable to suggest that in situ anisotropic permeability measurements (SOA Report, Section 4.2.8) coupled with two-and three-dimensional consolidation theories (Section 2.2.5) might give a rational explanation of the observed settlement rates. Also the paper presents little data regarding the precise stress history of the deposit and the magnitude of the stress increments compared to the degree of precompression. This information is

certainly needed to help explain a reported final consolidation settlement of only 0.17 m after 6 months compared to 0.53 m predicted to occur after 100 years based on laboratory tests.

6. MISCELLANEOUS

Some of the 15 papers included herein contain very interesting and/or practical information on a variety of topics. The first five papers discuss very diverse subject matters; the remaining papers primarily deal with unusual soils and special construction materials.

Alimi et al. present interesting results regarding the adherence between soil and reinforcement in reinforced earth. By means of in situ tests, they investigated the effects of cross-sectional area, length, thickness and inclination of the reinforcement and the influence of the density and depth of sand. From laboratory direct shear test data with a horizontal failure plane at the soil-reinforcement interface, the authors conclude that the laboratory approach underestimates the in situ adhesion due to the neglect of such factors as soil dilatancy characteristics and shape of the reinforcement. This paper should prove of value to workers in the area of reinforced earth.

Stresses developed in a cohesive soil during pile driving are analyzed by Massarsch and Broms using cavity expansion theory for an isotropic elastoplastic material. The theory predicts development of negative tangential effective stresses in normally consolidated clays during pile driving, which in turn suggests formation of cracks similar to those resulting from hydraulic fracturing. Such a phenomenon would temporarily cause a rapid dissipation of excess pore pressures immediately after pile installation. Pore pressures measured during pile driving in two soft sensitive clays exceeded the close-up pressure obtained from hydraulic fracturing tests, with a subsequent temporary rapid pore pressure dissipation. Thus the theoretical predictions, though simplified, are supported both by experimental data and field observations of performance of friction piles in clay. It should be noted that this phenomenon might also have an important effect on the short term performance of driven sand drains.

The paper by Akagi deals primarily with the undrained strength (c_u) of soft clays stabilized by installation of driven (displacement type) sand drains. Based on 12 case histories, the average in situ c_u is reported to be reduced to 60-80% of the undisturbed value immediately after drain installation, the amount of reduction not depending on drain spacing or soil sensitivity (considered quite surprising by the General Reporter). The author proposes a model for predicting the subsequent gain in c_u with consolidation which assumes w vs. log c_u to be parallel to the w vs. log σ'_{vc} curve obtained from oedometer tests on undisturbed samples. The total volume change is considered to have two components: that due to the volume of sand drains installed (which reportedly occurs quite rapidly in most cases); and that due to vertical settlement under the preload which apparently is little affected by disturbance due to drain installation. Unfortunately only two of the 12 case studies could be described in the paper so that it is difficult to assess the extent to which these findings can be applied to other sites.

Mitchell and Wan report the results of electro-osmotic consolidation tests on normally consolidated kaolinite and develop a general mathematical theory for the process. In contrast to earlier work, wherein simple theory underpredicts the amount and overpredicts the rate of electro-osmotic consolidation, this theory considers variations in both water and electrical flows. However experimental techniques to evaluate the parameters required for the model remain to be developed. The effect of electro-osmosis on clay behavior via induced negative pore pressure increments is shown to be identical to that resulting from consolidation due to direct loading, both having the same effect on changes in void ratio and undrained shear strength. After removal of the electrical field the clay is overconsolidated with respect to the in situ effective stress in the same way as if a physical load has been removed. This comprehensive paper should be of particular value to engineers interested in electro-osmotic stabilization of soft clays.

Tanimoto and Noda present results from acoustic emission measurements taken during CID triaxial compression tests (both stress and strain controlled) on well graded sands. Changes in emission count as a function of stress level, confining stress, strain rate and time are then correlated to the total elapsed time to failure (t_f). This is an interesting and perhaps promising approach, though the authors' conclusion that t_f can be predicted by emission count data regardless of soil type would appear somewhat optimistic.

Unusual Soils and Special Construction Materials

Anagnostopoulos and Papadopoulos investigate the shear strength anisotropy of a brown coal deposit, suggesting a means of representing the shear strength on any plane under consideration by use of polar coordinates. They use the procedure to present and evaluate the results of direct shear and triaxial compression tests on samples oriented in different directions.

The comprehensive paper by Blight et al. treats an unusual subject, that of employing pumped cemented slurries of mine tailings to prevent closure of very deep stopes (tunnels) used to mine gold in South Africa. The authors present strength data on silt size tailings treated with various combinations of portland cement, flyash, lime, etc. and compressibility data from model tests on these same materials with and without

different types of metal reinforcement (some similar to reinforced earth). The laboratory research and a subsequent prototype field test showed that pumped cemented slurries can be successfully used as fill for mining excavations.

Clayton studies the properties of chalks from a wide (but unspecified) variety of biostratigraphical levels and geographical locations in England for the purpose of assessing their suitability as compacted fill material. Though the LL of these pure chalks (typically more than 95% $CaCO_3$) varied between 22 and 32%, the PI was essentially constant at 6±1%. The author concludes that these material are trafficable (c_u>50 kN/m^2 from UU tests or CBR>10%) if compacted at a water content 3 to 5% less than the LL and presents data indicating that the increase in strength of compacted chalk with time may in part be due to increased values of the effective cohesion intercept (c'). The paper has practical value for engineers working with these soils.

Using conventional triaxial compression tests, Dupeuble and Habib investigate the effects of confining stress and strain rate on the strength-deformation behavior of cement grout mixes having varying proportions of sand-gravel, natural clay, bentonite and water. They discuss qualitative trends as a function of the above variables and describe the grout cut-off materials used in four dams.

The paper by Mikheev et al. presents the "first" field investigation of the settlement behavior of gypseous soils. Long term plate load tests were performed at three sites, the soils having very low natural water contents (2-9%) and 10 to 55% gypsum. Immediate settlements were small, but long term wetting produced greatly increased compressibility of these soils due to a combination of leaching out of gypsum and mechanical piping within a zone up to 1.5 times the width of the loaded area.

Minkov et al. describe the physical characteristics and subsidence-deformation properties of loess soils in Bulgaria, commenting on the anisotropy of the material and the basis used for developing zonal maps of loess deposits in the country. They discuss the design of foundations on loess and describe an approach involving use of a soil-cement cushion or mat 1 to 3 m thick which has been widely employed for major structures including nuclear power plants and high television towers. The authors also discuss the most appropriate type of stress distribution assumption to use when predicting settlements and the effects of adjacent loaded areas.

Ohta and Hata investigate the undrained shear strength of a compacted silty sand (non-plastic?) via constant volume direct shear tests. Void ratio-log static consolidation stress (σ_c') curves were first established as a function of initial water content (w_m), and subsequent CU direct shear tests showed a more or less unique c_u=0.3 σ_c' independent of w_m and hence e. Samples were then compacted dynamically and from the known w_m and e, the equivalent static consolidation σ_c' determined and multiplied by 0.3 to yield the predicted c_u. These values agreed reasonably well with measured CU test data on the compacted samples. Thus variations in molding technique, and hence soil structure, apparently had little influence on the strength of this soil, though the stress paths during shear were significantly altered.

Ordemir et al. describe the potentially expansive, fissured Ankara clay and present experimental data which reportedly support use of the Double Oedometer test procedure (the General Reporter questions this conclusion since it required considerable "translation" of one of the curves). Two case studies involving expansive Ankara clay deposits are discussed along with the construction procedures devised to prevent harmful swelling.

The paper by Rico et al. concerns gradation specifications for base course materials and reports triaxial strength data on a crushed basalt as a function of grain size distribution. The authors detected an "ideal" gradation (i.e. yielding maximum strength and rigidity) for their material, but conclude that it would have limited practical value due to sensitivity of the results to minor deviations from the ideal.

Togrol and Ersoy summarize laboratory data on the stress-deformation characteristics of a frozen compacted clay over a temperature range of -0.5 to -10°C. As would be expected, the undrained strength and modulus were highly dependent upon test temperature. Limited results illustrating the effects of strain rate and cyclic loading are presented.

7. CONCLUDING COMMENT

The General Reporter's evaluation and discussion of the 73 papers submitted to Session 1 no doubt contain some misunderstandings and/or errors in interpretation. Where this occurred, the authors were requested to send written comments to the General Reporter so that appropriate corrections could be made in the final report.

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