PORE PRESSURE MEASUREMENTS IN THE FIELD AND IN THE LABORATORY
MESURE DE LA PRESSION INTERSTITIELLE IN SITU ET EN LABORATOIRE

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INTRODUCTION

In presenting our report on this Specialty Session we have thought it best to include the introductory notes, which were in effect an abbreviated State-of-the-Art Report, together with the summaries of the verbal contributions to the discussion, using as far as possible the speakers' own presentations where these have been submitted to us in writing.

All the references are listed alphabetically at the end of the report.

It was requested that a fuller version of the contributions should be prepared and made available. Notification of how to obtain copies will be given in Geotechnique and in the News Letter of the Soil Mechanics and Foundation Division of the ASCE.

PORE PRESSURE MEASUREMENTS IN THE FIELD

Reported by P. R. Vaughan
(Imperial College, London)

Prior to the session an introductory note was circulated to those interested. This note and the introductory remarks are summarised, with references to the discussions at the session. The various contributions to the discussion are then summarised.

GENERAL PRINCIPLES OF FIELD PORE PRESSURE MEASUREMENT

For most pore pressure measurements, a piezometer is installed which consists of a small cavity separated from the soil by a porous filter. The filter must be strong enough to keep this cavity open. The fluid in this cavity connects with the soil pore fluid through this filter. The pressure of the fluid in the cavity is measured. The problem can be considered in two parts. Firstly, the pressure in the fluid in the piezometer cavity is presumed to be equal to the pressure in the soil pore fluid. In partly saturated soils containing two pore fluids at different pressures, equalisation with either pore pressure may be presumed. Secondly, the pressure in the piezometer cavity must be measured and generally transmitted to some point remote from the point of measurement. These two problems will be considered separately.

If pore fluid tensions exist greater than one atmosphere then indirect methods of measurement must be used.

EQUALISATION OF SOIL PORE PRESSURES WITH THE PRESSURE IN THE PIEZOMETER CAVITY

In saturated soil, with water in the soil pores and in the piezometer cavity, it is generally assumed that the two pressures become equal after a time lag determined by hydraulic considerations (Gibson: 1963). A similar assumption is made in the laboratory, and thus field and laboratory data are interpreted on the same basis. As indicated in the discussion by Richards, this assumption may not be valid always.

Unsaturated Soil

Two conditions may exist. Firstly, the air may exist in continuous passages through the soil, in which case it is generally at atmospheric pressure. Such conditions exist in soil profiles above the water table when problems of heave and collapse may be considered. The relevant measurement is then the suction due to capillarity in the pore water. Direct measurement of this suction is limited by cavitation. Indirect methods of measurement, generally on samples removed from the ground, are commonly used. Methods of making such measurements in-situ were described by Richards and Escario.

Secondly the pore air may exist in isolated pockets in the fill, and both the pore air and pore water pressures may be positive. This situation typically exists in fills of low permeability placed wet of optimum, where pore pressures are of engineering significance in controlling stability. Due to the slow rate of diffusion of air through water, equilibrium between the air and water phase in non-homogeneous soil may be reached slowly. (Barden & Sides: 1967) The two phases will be at different pressures, the air pressure being the higher. For a complete definition of the problem, the effective stress principle for such a two-phase system must be understood and both pore air and pore water pressures must be measured. For engineering purposes it may be enough to establish the likely errors arising from an uncertainty as to which of the two is being measured in the piezometer cavity (Torblaa: 1966).

It has been postulated that the pressure in the piezometer cavity, initially equal to the pore water pressure, will tend to equalise with the pore air pressure, either rapidly by direct communication if the piezometer filter has coarse pores and an air entry value less than the difference between the air and water pressure, or slowly by diffusion if it has fine pores and a high air entry value' (Hilf:1965, Bishop et al: 1964, Scott & Kilgour:1967). This diffusion rate is relatively rapid if the pore water pressure is negative, and much slower if it is positive. If diffused air can be flushed from the piezometer cavity, continuity with the pore water can be re-established. This is relatively simple with piezometers with high air entry filters. A temporary re-establishment may occur with low air entry filters, if the pore water pressures are positive (Bishop et al:1964). Piezometers with high air entry value filters and a flushing system are the only ones where continuity with the pore water can be reliably re-established. However, initial diffusion of air through a high air entry value filter may be retarded by using a thick filter, thus allowing a piezometer without a flushing system to measure pore water pressures for a longer period.

Where pore pressures are positive, it is the pore water pressure which primarily controls effective stress, and where rates of drainage and consolidation are being observed, it is the pore water pressure which responds systematically to the external drainage conditions. Thus, if the pore water and air pressures are significantly different, the minimum requirement is that pore water pressures should be measured. It is generally assumed that in permeable soils, or when pore pressures are high, the errors involved in inadvertently measuring pore air pressure rather than pore water pressure are not significant. This was confirmed by discussion at this session.

In clay soils with $K<10^{-7}$ cm./sec. approx. it has been shown that at low pore pressures the inadvertent measurement of pore air pressure can give anomalous results. (Bishop et al:1964, Little:1964)

Some data showing comparisons between different types of piezometers which may be expected to measure pore air or pore water pressure has been published (Torblaa:1966 , Bishop et al:1964, Little:1964, Barge et al: 1964, Pinkerton & McConnell:1964). Additional information was given in discussion by Post and Picaut, Buck and Nonveiller. The contribution of Buck indicates that these anomalies may arise with organic foundation materials (presumably due to the presence of gas) as well as in compacted fill.

The other discussions confirm the general situation for measurments in partly saturated soils, which may be summarised as follows. The filter must have an air-entry value greater than the air-water pressure difference in the soil if pore water pressures are to be measured. If, with such filters, the pore water pressure is negative, air diffuses into the piezometer cavity quite rapidly and the piezometer starts to record pore air pressure. Continuous measurements of suctions can only be made if there is provision for 'de-airing' the piezometer cavity. At high pore pressures the air-water pressure difference is small and of no engineering significance. At intermediate pore pressures where both air and water pressures are positive but significantly different, as may occur in a dam when construction pore pressures dissipate to steady seepage values, then with high air entry value filters there is some evidence that with both hydraulic piezometers (Bishop et al:1964) and sealed electrical piezometers (Torblaa:1966 and discussion by Post and Picault) pore water pressures are recorded for a considerable period of time without the need for 'de-airing' to re-establish continuity with the pore water. Further information is clearly required to establish the conditions in which 'de-airing' is required and hydraulic piezometers must be used and those in which 'de-airing' is not required and sealed cavity piezometers with pressure transducers may be used if desired.

A further uncertainty in the measurement of pore pressure in wet compacted fills arises from the tendency of all types of piezometers to record pore pressure in excess of overburden pressure when this pressure is small. Pressures just greater than overburden pressure have been observed with hydraulic piezometers with flushing facilities and high air entry value filters (McLaren:1960). Pressures up to twice the overburden pressure have been observed with electric piezometers without flushing tubes, but with high air entry value filters. Thus this effect cannot be attributed entirely to the inadvertent measurement of pore air pressures. Such observations can have engineering significance in low fills. Possible explanations are given in discussions by Richards and Juarez-Badillo.

MEASUREMENT OF THE PRESSURE WITHIN THE PIEZOMETER CAVITY

With piezometers installed in boreholes which are subsequently accessible, a simple 'Casagrande' standpipe can be used and a direct reading of pressure obtained. Since bubbles of air rise in the standpipe such piezometers are 'de-aired' automatically. Otherwise, as with piezometers built into or under fills, a remote reading system must be used. Either the piezometer cavity is connected by tubes to an external measuring point, or the pressure is measured at the piezometer cavity. In the first type, two tubes are used, so that air can be flushed from the cavity. The tubes are generally of plastic and the permeability and chemical stability of the plastic must be considered (Daehn : 1962. Bishop et al:1964 and discussion by Little). Piezometers of this type were first used by the USBR, and they have been modified with high air entry value filters for use in unsaturated fills (Hilf:1956, Bishop et al:1964).

Advantages of these piezometers are (1) Continuity with the pore water can be re-established in partly saturated soil. (2) Pressure measurement is external, calibrations can be checked and there are no moving parts to give trouble in long term operation. (3) The operation of the piezometer can be checked in-situ. (4) The piezometer can often be used for measurement of in-situ permeability. This may be of considerable value where measurement of rates of drainage is involved (Gibson:1963, Bishop et al:1964, Rowe:1968, Proc. Conf. on in-situ investigations in soils and rocks, Section V, British Geotechnical Society:1969) Developments in the theory for interpreting such tests are discussed by Gibson.

Disadvantages are (1) Cavitation within the connecting tubes restricts the relative elevations of the piezometer tip and the measuring point. When the tubes are connected to the top of a standpipe, air bubbling systems may be used to meet this difficulty (Cooling:1962, Conté & Chanez: 1964, Vaughan:1965). (2) De-airing and pressure measuring devices may require large instrument houses. (3) De-airing may be required, every few weeks if suctions are being measured and at intervals of several years at positive pressures. (4) Hydraulic systems are subject to freezing in cold climates. (5) Blockages may occur if clean water is not used.

In the second type of instrument, the piezometer cavity is sealed by a diaphragm. Two systems of measurement are used. Firstly, the pressure on the diaphragm is recorded by an electrical transducer and transmitted to the measuring point. Either the space behind the diaphragm can be vented (Cooling:1962, Arhippainen:1964), in which case the datum pressure is controlled and the response of the instrument to changes in the datum pressure can be observed in-situ, or it is sealed in the instrument and neither datum nor calibration can be checked after installation. In discussion Collins described a new type of electric piezometer which can be calibrated in-situ. Vibrating wire (Torblaa:1966, Bishop et al:1964, Cooling:1962) and differential transformer (Arhippainen :1964) pressure gauges have been used. Secondly, the diaphragm may operate a valve in a circulating system, which opens when a pressure is applied which balances the pressure on the diaphragm. An air circulating system (Warlam & Thomas:1965, Civ. Eng.:1967) or a hydraulic system (Lauffer & Schober:1964) can be used. With such systems, pressure measurement is external and accessible although the diaphragm and valve must be calibrated before installation.

Advantages of sealed cavity piezometers are that the external measuring systems can be made compact and portable, and that no large gauge houses are needed. They are not generally subject to freezing and the level of the measuring point and the connections to the piezometer can be above the tip and the piezometric pressure level without problems of cavitation arising.

Disadvantages of such systems are that they cannot be 'de-aired' to re-establish continuity with the pore water and they involve, to a greater or lesser extent, calibrations which cannot be checked after installation and inaccessible electrical or mechanical moving parts which may give trouble in long-term operation.

In principle, it might be anticipated that the long term reliability of piezometers would be greater the simpler the inaccessible part of the equipment is. On this basis hydraulic piezometers should be the most reliable and the reliability of the sealed cavity type of piezometer would decrease with increasing complexity of the equipment. Little data on long term reliability of equipment and comparative measurements has been published. A number of contributors to the discussion gave such information (Gibbs and Daehn, Little, Post and Picaut, Sherman, Nonveiller, Gadsby, Wolfskill and Japelli). Some broad conclusions may be drawn from this information concerning the relative reliability of hydraulic and electric equipment. Hydraulic equipment has been in operation successfully for 30 years. Electric equipment has operated apparently giving correct readings for about 15 years. Major failures of both types of equipment have occurred, although with hydraulic equipment the type of failure is more readily identified. Excluding the major failures, both types of equipment have performed well, and on this limited data the hydraulic equipment is more reliable by only a small margin. Developments in electric equipment may have reduced this margin. Success with both types depends on quality of manufacture and installation. Probably the former is more important with electric equipment and the latter with hydraulic equipment.

RESPONSE OF PIEZOMETERS TO PORE PRESSURE CHANGES

Theoretical expressions for the hydraulic response of piezometers (Gibson: 1963) and experimental data (Penman:1960) are available. Most systems have a response rate adequate for engineering purposes. An exception is the use of open standpipe piezometers in soils of low permeability. Methods of overcoming this problem by temporarily locating a rapidly responding electric pressure transducer in the standpipe below the water level with a packer have been devised (Lundgren:1966, Brooker et al:1968). The simplicity and reliability of the standpipe is retained and the transducer can be checked and calibrated at any time.

Pore pressure changes under dynamic loading were not considered at the session. The increase in response time due to silting up of the piezometer was described by Sherman.

INSTALLATION OF PIEZOMETERS

Piezometers are generally sealed into boreholes or built into fill as it is constructed. The errors which may occur in a borehole piezometer reading if the borehole is inadequately sealed have been analysed (Vaughan:1969). The seal permeability can be higher than that of the soil without significant error. In some circumstances this filter can be omitted and the piezometer unit placed directly in the grout filled borehole (Vaughan:1969). With this system a large number of piezometers can be placed in one borehole.

SUMMARY OF DISCUSSION AT THE SPECIALITY SESSION

The summaries by Gibbs & Daehn, Little, Post & Picaut, Nonveiller, Sherman, Escario, Juarez-Badillo and Gadsby have been provided by the contributors. The remaining summaries have been prepared by the reporter.

H. J. Gibbs & W. W. Daehn (U.S.B.R., Denver, Colorado) discussed Bureau of Reclamation experience with field measurements of pore pressure.

The Bureau of Reclamation has had more than 30 years experience with field pore pressure measurements. The first installation of the hydraulically-operated twin-tube piezometers dates back to 1939 at Fresno Dam in the State of Montana and , although the installation has been reduced from three sections to one section when the terminal facility was rehabilitated within the past year, none of the piezometers were lost due to malfunction. The piezometer system as presented in the latest publication of the Earth Manual involves the same basic principles as originally conceived, but materials and installation procedures have been continually improved over the years. One such improvement has been in the use of ceramic filter discs, proposed by J. W. Hilf (1956). tried on an experimental basis, and now considered standard. These ceramic discs provide much quicker response in pressure readings and give an indication of negative pore pressures.

Considerable reliance has been placed on the performance of these hydraulic piezometers, both during construction of major earth embankments and during their operation. Factors considered in their use include relative economy and availability of materials, ease of installation, and duration of readings. The simplicity of reading procedures lends itself to the quality of manpower available for obtaining continuing periodic readings in later years.

Although there is much interest in newer proposals for obtaining pressure measurements by more sophisticated instrumentation, the use of hydraulic piezometers has been continued because of the reliance placed on the readings and the serviceability of the present installations. From a total of 920 piezometers installed on 18 different dams since 1942 where PVNC plastic (Saran) tubing was used, 873 piezometers or 95 percent are still operational.

A. L. Little, (Binnie & Partners, London) discussed the reliability of piezometer installations in dams.

A total of over 1000 piezometers had been installed in 22 dams with which Mr. Little had been associated, exclusive of simple stand pipe (Casagrande) instruments. A total of 10% of the hydraulic instruments had gone out of operation for various reasons.

The earliest installation was at the Usk dam (Penman; 1956, Sheppard & Little: 1955) where 18 twin tube hydraulic piezometers were installed between 1951 & 1954, together with 9 steel standpipes which were subsequently capped and connected to twin tubes. In 1966 when the installation was shut down, 4 twin tube and 4 standpipe piezometers were inoperative.

Hydraulic piezometers at two dams had shown high failure rates. At Treweryn dam Nylon 6 was used for the tubes, and they became blocked by soluble material contained in the nylon and redeposited in the tubes. At the Shek Pik dam in Hong Kong, where polythene tubes were used, 26 out of 74 hydraulic piezometers were now inoperative. The core of this dam had cracked and this was attributed to large differential movements.

At the remaining 20 dams only 38 failures had occured, a rate of 4¼%. There was no evidence that the age of the installation had any influence on the failure rate, most failures occurring shortly after installation, often due to damage by plant.

A total of 172 vibrating wire electrical piezometers had been installed, of which 7% were no longer working. However, the accuracy of most of these instruments could not generally be checked. At one site (Little:1964) pairs of electric and hydraulic piezometers were installed in a clay fill in which suctions existed. The electric instruments, with low air entry value filters, gave anomalous results.

R. Post & J. Picaut (Coyne et Bellier, Paris) discussed the comparative performance of electric & hydraulic piezometers.

Three earth-rock dams (Serre-Poncon, Mont-Cenis and Saint-Cassien) built by Electricite de France, have been selected where pore water pressures are being measured both by electric and hydraulic cells, for the purposes of comparison of the two systems.

This has led to the following conclusions:

(1) Pressures as measured by electric cells usually differ by one to two metres of water exceptionally up to 5 metres. This discrepancy remains approximately constant with time, with a tendancy to decrease, the electric reading being generally but not always higher. For both types of instrument scattering around the mean curve is in the range of one to two metres of water.

(2) The proportion of instruments out of order at the present time (after 12 years of measurement at one dam) is very small for both types, being 5% on average, for a total of 121 electric cells and 80 hydraulic cells. Damage always occurred on installation or shortly afterwards.

(3) The use of stones with high air entry values for both types of cell as compared to the use of coarse stones did not affect the measured pore pressures in the core during construction. But all these core materials were of low or medium plasticity and compacted with a moisture content above the standard optimum.

The use of high air entry stones reduces scattering of the readings of the hydraulic cells and permits the measurement of negative pore pressure. With the electric cell the negative pore pressure can be measured only during a limited time after placement, because of air diffusion through the stones.

(4) Both installation and measurement appear to be easier and more rapid with electric cells. Moreover, these can be placed at any level in relation to the reading station.

(5) Cost, including installation, is in the same range for both types of instrument.

W. C. Sherman (WES, Vicksberg) discussed the long term performance of piezometers.

The US Army Engineering Waterways Experiment Station (WES) has been concerned with the relative merits of piezometers for many years. In 1949 an experimental piezometer system was installed to evaluate various types of devices for measuring positive pore water pressure in a backswamp deposit of fat clay adjacent to the Mississippi River. The installation consisted of seven different open-system type piezometers in duplicate and a single electrical type piezometer. The piezometers were observed for a period of about two years and then left unattended until 1967 when observations and time lag determinations were made and a number of the piezometers retrieved. Some of the conclusions of this study were: (1) drive point piezometers are vulnerable to damage and clogging during installation, (2) an increase in basic time lag can be expected over an extended period of time due to clogging of the sand filters and porous pickups, (3) extensive corrosion of metal parts occurred in areas of acidic soil, and (4) several piezometers including the Casagrande and wellpoint piezometers had a relatively long useful life despite the increase in basic time lag.

Similar systems of more recent types of commercially available piezometers were installed by WES in 1967. The devices included the hydraulic, pneumatic and electrical transducer types of piezometers. The piezometers were installed to measure positive pore water pressure at three different sites. The groundwater conditions at the sites were as follows: (1) piezometric level varying with time as a result of rising and falling river stages, (2) piezometric level above ground surface slowly dissipating with time, and (3) a relatively stable piezometric level below ground surface serving as a controlled condition. Observations are being made at frequent intervals to determine the accuracy and reliability of the various devices over extended periods of time.

E. Nonveiller (Institut Geotechnique, Zagreb, Jugoslavia) discussed the long term behaviour of vibrating wire piezometers.

In the Lokvarka Dam (Yugoslavia), on which I have reported on the IVth ICSMFE in London, the clay core was instrumented in two cross sections for the measurement of pore pressure. Piezometer tips were installed in six horizons on a total of 15 measuring points, each provided with two piezometer tips. One cross section was instrumented with hydraulic type cells the other one with vibrating wire instruments (Officine Galileo, Milan). The vibrating wire instruments were operating from the time of installation, while considerable difficulties had to be overcome in order to make the hydraulic tips operative. They never worked satisfactorily due to some technical details which always caused trouble.

From the 30 vibrating wire cells installed during construction 27 were operating at end of construction, 21 three years later and 20 are working today, 14 years after installation.

In order to check the reliability of the readings the ratio of pore pressure v. hydraulic head on the elevation of the tips is compared for the upstream row of tips, 3,0 m from the boundary of the core on three of the lower horizons. They are:

Horizon	ratio u/p	
	1958	1969
II	1,0	1,07
III	--	--
IV	1,00	0,95
V	0,825	0,86
Average	0,96	0,96

In the cells near the upstream face of the core where the fill is most probably completely saturated a ratio $u/p = 1,0$ is normal. The actual ratio was near this value in 1958 as well as in 1969. This evidence confirms that the pore pressure readings obtained by the vibrating wire piezometers are reliable during long periods of time.

The type of cells used in the Lokvarka dam had no means to prevent air entering the tips, since at that time the advantages of ceramic tips were not known. On some tips installed in the central and downstream part of the core, pore pressures in excess of the theoretically expected values are read, which is probably the pore air pressure in the nonsaturated part of the core. With ceramic filters this inconvenience would be eliminated.

The simplicity of installation and the ease of taking readings is a considerable advantage of this type of instrument compared to the hydraulic piezometers. The evidence in the Lokvarka dam shows that the vibrating wire piezometers can give satisfactory trouble-free operation over periods of time sufficient to evaluate the behaviour of earth dams and to make all measurements necessary to verify the design assumptions as well as to collect scientific data.

J. W. Gadsby (C.B.A. Engineering Ltd., Vancouver) discussed the behaviour of diaphragm piezometers in the Hugh Keenleyside Dam (formerly the Arrow Dam)

The Arrow Dam was constructed by placing portions of the dam under water. The lower till core was placed under water (Golder & Bazett:1967, Casagrande, Golder and Bazett:1969, Bazett:1970). The upper till core was a conventional rolled fill.

The geometry of the dam and method of construction required that the gauging point for the piezometer cells in the till core would be considerably higher than the piezometric surface and this dictated the types of instrument which could be chosen. It was decided to group the instruments on planes transverse to the axis of the dam: one with pneumatic type (Warlam) cells and the other with electrical vibrating wire type (Maihak) cells. At some locations in both planes, the two types were twinned to provide a check on the readings.

It was known from large scale field tests that the till core would be partially saturated and that negative water pressures and, possibly, high positive air pressures would exist. The lower till core was initially at about 50% saturation and the upper rolled till core about 90%. Coarse porous filters, with a low air entry were deliberately chosen in order that the cavity in the piezometer would be easily re-wetted.

Twinned instruments 130 (Maihak) and 131 (Warlam) approximately 10 ft. apart in the lower core showed systematic response to changes in headpond level with a slight time lag. Initially agreement was within 1 ft. of water. As the headpond level increased the difference increased to 8 ft. the Warlam piezometer being the higher and rising temporarily above headpond level. At the end of impounding the difference decreased to 3 ft. in an average recorded pressure of 85 ft.

Twinned instruments 139 (Maihak) and 112 (Warlam) approximately 10 ft. apart in the upper compacted core initially showed small positive pressures with the Maihak reading higher by about 7 ft. of water. This difference tended to decrease when the headpond level rose and at the end of impounding the difference was 5 ft. on average, in a pressure of 50 ft. Systematic response to headpond level was observed.

Piezometric readings have not yet been obtained through an annual cycle of filling and drawdown.

It is believed that the piezometers are responding accurately, although the piezometers installed in the lower core are likely to have been dewatered due to the negative pore water pressures existing in the soil at the time of installation. The readings show consistent behaviour for both types of diaphragm piezometers.

The pore pressure responses reported do not necessarily describe the behaviour of the entire till core.

V. Escario (Laboratorio del Transporte y Mecánica del Suelo. Madrid) described a method of insitu-measurement of pore water tensions.

A method is described which can be used for practically any value of the negative pressure.

Referring to Fig. 1 the cell consists of a porous element (gypsum plaster in the tests performed) which will be called the "sensitive element". The pore water in this element becomes in equilibrium with the soil. The contact with the ground is made through the cylindrical surface of a fine grained porous stone. An intermediate ring is placed with another porous stone; it is separated from the sensitive element and from the lower stone by means of semi-permeable membranes.

The sensitive element has two electrodes with which the conductance is measured as a first step. This value is only used as a zero or reference point.

Water at atmospheric pressure is supplied to the intermediate ring. Air pressure is applied above the sensitive element; if its value is equal to the suction, no migration of water will be originated in the sensitive element. By successive approximations, moving around the zero conductance determination, the right value of the air pressure is obtained.

The measurement takes from half to one hour. The errors checked were below ± 10% and most often below ± 5%.

E. Juarez-Badillo (National University, Mexico) discussed a possible explanation for the observation of construction pore pressures greater than overburden pressure.

A probable qualitative explanation of temporary pore pressures - sufficiently high to cause pure effective tension in the soil structure is offered in terms of the concepts of "fundamental stress σ fund" and "stored pressure σ_s", defined elsewhere (Juarez-Badillo:1969), as follows:-

In a wet compacted fill the clay particles have a highly oriented structure. As time elapses, the thixotropic properties of the clay tend to decrease the degree of orientation of the particles under constant volume conditions tending towards an at random structure. During this process the pore pressure increases and the "stored pressure σ" is built up. In a preconsolidated saturated sample under isotropic stresses $\sigma_s = \sigma_e - \sigma_c$ where σ_e and σ_c are the "equivalent consolidation pressure" and the "consolidation pressure" respectively; in this condition:

$$\sigma \text{ fund} = \sigma' + \sigma_s = \sigma - u + (\sigma_e - \sigma_c)$$

where σ' is the effective isotropic stress.

Returning to the compacted fill, the building up of σ_s and u may possible turn σ' negative with the only condition that σ fund be always positive. The fundamental stresses σ fund are assumed to represent the "real stresses conditions" among the clay particles. Thus, nevertheless the effective stress is negative that does not mean that "pure tension" is present in the clay structure.

B. G. Richards (C.S.I.R.O. Australia) described briefly the psychrometric technique for measuring soil water tension, which has been developed during the 11 years since C.S.I.R.O. started making such measurements. (Richards: 1969) The technique measures total soil water potential and has been developed both for the laboratory and for insitu field measurements. The technique depends on the thermodynamic relationship between the free energy of the soil moisture and the relative humidity in the adjacent air. The humidity in the space in the piezometer probe is determined by a thermocouple, one junction of which can be made to act as a wet bulb by initially depressing its temperature below the dew point by passing a current through it. The probe is calibrated against standard sodium chloride solutions. Constant temperature conditions at the probe are required. The probes can be miniturised such that four can be installed in a 3 in. long x 1½ in. dia. triaxial sample. An accuracy of ± 2 lb/in.2 or ± 3% (whichever is the greater) can be achieved in the laboratory. With the standard field equipment the figures are ± 15 lb/in.^{2} or ± 10%. Measurements can be made at approx. 15 min. intervals. The equipment is compact and cheap.

Mr. Richards also commented on the possibility of the water pressure in the cavity within the filter element of a conventional piezometer being different from the pressure in pores of the soil. In Australian conditions with very fine grained soils differences of between ½ and 2 times were possible if the size of the soil pores and the salt content of the pore water were such that osmotic effects could develop. (Bolt & Lagerwerff:1965)

J. P. Collins (James P. Collins & Associates Cambridge, Massachusetts) described a new electric piezometer that can be re-calibrated in-situ. Mr. Collins prepared a paper which was available to participants.

The principle of this piezometer is shown in fig. 2. The pore pressure in the piezometer cavity (Pa) is transmitted through holes in the back-up block to a divider diaphragm. For normal pressure measurement the divider diaphragm lifts (by less than 0.001 in.) to rest against the pressure sensing diaphragm of the transducer, which measures the fluid pressure in the piezometer cavity. To calibrate the piezometer a gas pressure (Pc) is applied to the space between the divider diaphragm and the transducer diaphragm. When this pressure exceeds Pa the divider diaphragm seats on the back-up block and the transducer records the calibrating gas pressure. To calibrate the piezometer at pressures less than Pa a back pressure (Pb) is applied to the other face of the diaphragm. If $Pb = Pc > Pa$ the zero pressure reading for the piezometer is determined. Thus the incremental pressure calibration and the zero shift of the transducer can be determined at any time.

Mr. Collins went on to describe the adaption of this principle to a borehole piezometer installed at the base of a standpipe. A pressure sensing device which could be calibrated in-situ using the technique described above was lowered to seat with an 'O' ring gravity seal on the driven filter tip. If access to the top of the standpipe was possible, the pressure device could be removed and the tip de-aired with a vacuum device lowered in place of the pressure device. The transducer used in this particular device utilised a diffused solid-state strain gauge circuit, in which semi-conductor elements are diffused directly into the transducer diaphragm. The transducer also incorporates an analogue-digital converter producing a digital signal proportional to the applied pressure. This digital signal is unaffected by the resistance in the connecting cable, simplifying transmission of the signal over long distances. It also enables the pressure to be displayed or automatically recorded by simple and inexpensive equipment. A very rapid response is possible.

R. E. Gibson (Kings College, London) refered to the use of constant pressure seepage tests (either inflow or outflow) on hydraulic piezometers to determine the coefficients of permeability and consolidation or swelling of the soil adjacent to the piezometer. This technique was being used quite widely in Britain for obtaining additional useful data from hydraulic piezometer installations. Interpretation of these tests was usually based on Prof. Gibson's theory (Gibson:1963) for a spherical piezometer which assumed that pore pressures set up in the soil were due to seepage only. This gave the following equation:

$$q = \Delta u . \frac{k}{\gamma_w} . \pi . 4 . R . \left(1 + \frac{1}{\sqrt{\pi T}}\right)$$

where $T = \frac{c_v . t}{R^2}$

c_v = Coefficient of consolidation (or swelling)

R = radius of piezometer

Δu = out of balance pressure

k = Coefficient of permeability

q = flow into or out of piezometer

t = time elapsed from beginning of test

The permeability and coefficient of consolidation was obtained by plotting flow against $1/\sqrt{t}$, which gave a linear plot.

Experience had shown that permeabilities derived from the application of this equation were generally satisfactory but values of the coefficient of consolidation or swelling were often in poor agreement with observed field behaviour (See, for instance, the proceeding of the British Geotechnical Conference on in-situ testing of soils, 1969, Session E)

Professor Gibson had recently extended the theory to include the effects of the pore pressures set up in the soil by the immediate application of the water pressure at the boundary of the piezometer. This gave an equation of the form:

$$q = \Delta u . \frac{k}{\gamma_w} . 4 . R \left[3A + \frac{3}{2}(1 - A)\frac{1}{\sqrt{\pi T}} \right]$$

where A was the Skempton pore pressure coefficient.

Thus the flow was dependent on A as well as c_v and c_v could not be derived directly unless $A = \frac{1}{3}$. With other values of A a ficticious value of c_v would be obtained if the old theory were used. However, the plot of q against $1/\sqrt{t}$ remained linear and thus the value of k could be determined by this method with confidence. This was in agreement with field experience.

G. F. Buck (R. C. Thurber & Associates, Victoria, Canada) described comparative measurements made in the muskeg foundation of a road fill. Initially pneumatic diaphragm piezometers were installed and subsequently two standpipe piezometers were installed to check high pressures recorded by the pneumatic instruments. The pressures recorded in the standpipes were lower than those recorded by the pneumatic piezometers by up to 10 ft. of water, and, during the second stage of filling (which was done in three stages) the increases in pore pressure recorded by the pneumatic piezometers were in some cases greater than the increase in vertical load, whereas with the standpipe piezometers the increase in pore pressure was only 65% of the increase in load. The pore pressures recorded by the standpipes were consistent with effective stress calculations and field performance.

A. L. A. Wolfskill (M.I.T., Cambridge, Massachusetts) described a piezometer installation in a deep foundation strata of Boston Blue Clay, where both electric transducer piezometers and hydraulic piezometers were used. For six months the agreement between the two types was good but fourteen months later four out of six of the electric piezometers had failed, and comparison of the results of the remaining two with the hydraulic piezometers, which all remained operational, indicated that they were reading higher by 8 ft. of water (19%). This represented the worst experience of M.I.T. with electric piezometers to date. In their best performing installation a failure rate of 10% had occurred. Dr. Wolfskill said that M.I.T. experienced a failure rate with hydraulic piezometers of between 10% and 20%.

R. Jappelli (Italy) commented briefly on Italian experience in using Galileo type electric piezometers in some 20 dams. He commented that the experience of stability and durability of these instruments was satisfactory, but some anomalous results had been experienced where they had been used in plastic core fills.

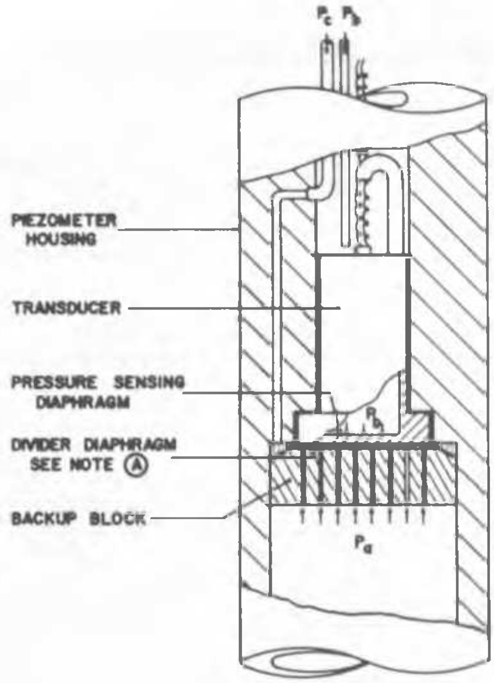

Figure 2
ELECTRICAL PIEZOMETER THAT CAN BE RE-CALIBRATED IN SITU. (J. P. COLLINS)

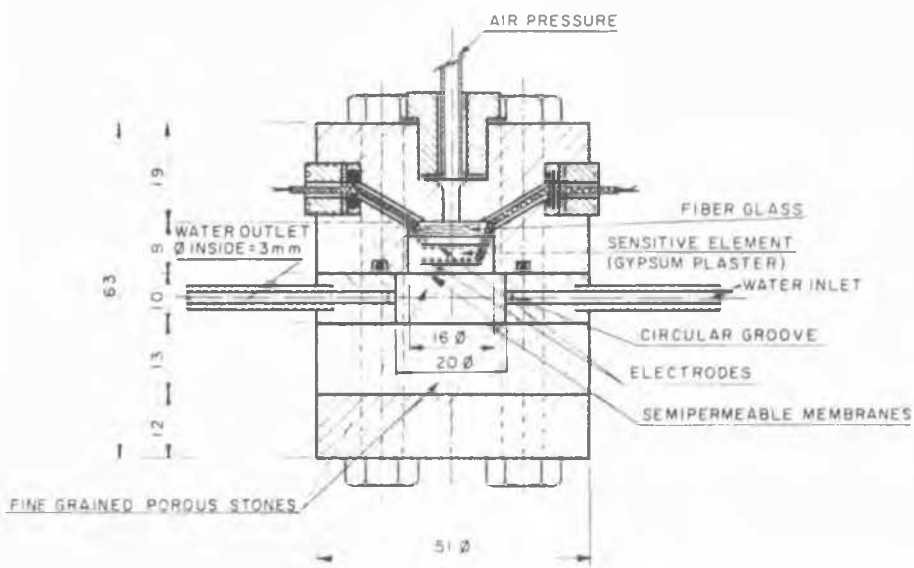

Figure 1
IN SITU PORE WATER PRESSURE CELL (V. ESCARIO)

PORE PRESSURE MEASUREMENT IN THE LABORATORY

Reported by Alan W. Bishop.

Professor Bishop presented briefly the Introductory Notes which had been prepared by Dr. G. E. Green and himself. Three speakers contributed to the verbal discussion. The summary of Professor Helenelund's discussion has been prepared by the reporter.

INTRODUCTORY NOTES BY PROFESSOR A.W. BISHOP AND DR. G. E. GREEN (IMPERIAL COLLEGE, LONDON)

INTRODUCTION

The principles involved in the laboratory measurement of pore pressure are basically the same as in the field measurement of pore pressure, but their relative importance may be very different due to major differences in time scale, pore pressure gradient, and the physical dimensions of the 'sample' in the two cases.

A striking example of this difference occurs in a paper by Whitman, Healy and Richardson (1961) which shows the importance of reducing time lag in observing the transient pore pressure occurring in a standard consolidation test. In contrast, the permissible time lag in the field is usually very much greater.

A detailed analysis of all the factors considered to influence the correct determination of pore pressure in the triaxial test has been given by Bishop and Henkel (1962), Appendix 6, to which reference should be made for a quantitative assessment of the relative importance of these factors.

Three distinct classes of laboratory problem may be distinguished:

1. Undrained tests, where ideally the pore pressure is uniform throughout the sample, but varies with time primarily as a result of controlled changes in total stress or strain applied to the sample.

2. Consolidation tests and pore pressure dissipation tests, in which controlled boundary drainage results in pore pressure gradients within the sample. In these tests the rate of change of pore pressure with time is a function of the coefficient of consolidation or swelling, and the drainage path.

3. Model tests (for example, model footings), in which the undrained pore pressure would, even ideally, be non-uniform. Here both redistribution of pore pressure and boundary drainage occur, and the rate of change of pore pressure is a function both of rate of loading and of the coefficient of consolidation and drainage path.

For each of these classes of problem it is necessary to establish that the volume factor of the null indicator or transducer and the intake area of the filter connected to it match the coefficient of consolidation, compressibility and the size of sample or model proposed, and the rate of testing.

UNDRAINED TESTS ON SATURATED SAMPLES

a) Under the application of an equal all-round stress the time for the equalization of the pressure in the null-indicator or transducer with the pore pressure in the sample is controlled by the basic time lag of the system. Table 17b of Bishop and Henkel (1962) shows that this time lag is not determined by the volume factor or compliance of the null-indicator alone, but is influenced radically by the compressibility and coefficient of consolidation of the sample.

For a soil of low compressibility and low coefficient of consolidation, the volume factor of the transducer and area of the filter become of critical importance. A small quantity of trapped air or a small leak can vitiate the results under these circumstances. The detection of these errors in closed systems, using only electrical transducers, would be a useful topic for discussion. For example, layouts used by Nash and Dixon (1960) and Barden and McDermott (1965) use the mercury null indicator (Bishop and Eldin, 1950) to determine the volume factor of the transducer and to detect whether air is trapped in it. These problems are also discussed by Morgan and Moore (1968)

With a sample of low compressibility and a transducer with a large volume factor and with trapped air, the equilibrium pore pressure after equalization can be significantly less than the true 'undrained' value. An expression is given by Bishop and Henkel (1962), p. 196, from which the magnitude of this error can be determined.

Observations of pore pressure under undrained conditions are complicated by the influence of temperature changes, which may be irreversible (Henkel and Sowa, 1963).

b) During the application of the deviator stress, the pore pressure observed at any point in the sample is additionally time dependent, due to the progressive equalization of non-uniformity in pore pressure within the sample, resulting from end restraint or from a natural tendency to zone failure (Bishop et al:1960 a,b; Olson:1960).

This non-uniformity may be reduced by using lubricated end plattens, though this complicates the measurement of pore pressure (and also the consolidation stage of consolidated undrained tests). Examples of the use of lubricated plattens are given by Blight (1961, 1965), Rowe and Barden (1964 a), Barden and McDermott (1965) and Schofield and Wroth (1968). Alternatively, and more simply, the pore pressure may be measured at the ends of the sample or over the whole surface area, and time allowed for the pore pressure to become substantially equal throughout the specimen. The theoretical time factors (due to R. E. Gibson,1963) and the experimental evidence supporting them are given by Bishop and Henkel (1962), p. 200. It is apparent from worked examples given in Table 18 of the above reference that except for small samples with a relatively high coefficient of consolidation, the time for 95% equalization is much greater than the test duration usually allowed for, and is indeed very large compared with the basic response time of a 'hard' transducer or null-indicator. This provides a strong argument for testing soils showing a peak to the stress-strain curve under a controlled rate of strain. Pore pressure observations during the post-peak runaway of a controlled stress test are likely to be in error, however 'hard' the transducer. Concentration on basic response time may mislead the investigator into overlooking the pore pressure gradients within the sample, which can lead to very significant errors in the observed shear parameters.

For larger samples (it may be noted that samples up to 1 ft. dia. x 2 ft. high appear to be necessary to define the strength parameters of stiff-fissured clays) the time necessary for equalization is quite impracticable. The best procedure is then the use of probes to measure the local unequalized pore pressure. This procedure was introduced by Taylor (1944), using a single probe inclined in the probable shear zone of a small sample. Difficulties in technique and problems of sample disturbance and stress concentrations are greatly reduced in large samples where the probe is relatively much smaller in size.

Probes have been used in 4 in. dia. samples by Blight (1961 and 1963 a,b) and Barden and McDermott (1965), in 8in. dia. samples by Tschebotarioff et al (1956) and in 12 in. dia. samples by Hall and Gordon (1963). In all cases the samples were compacted or remoulded. A discussion on the use of probes in large undisturbed samples subject to zone failure would be of considerable interest.

The measurement of the transient pore pressures set up in tests involving impulse or repetitive loading presents the above problems in their most acute form (Johnson and Yoder, 1963; Peters 1963). The meaning of such measurements deserves special consideration.

c) The relationship between observed pore pressure and applied stress may be influenced by the rate of testing, not only due to (i) time lag in the pore pressure device and (ii) progressive equalization of non-uniformity in pore pressure resulting from end restraint or from a natural tendency to zone failure, but also due to a genuine modification in the behaviour of the soil structure as the rate of shear is reduced. This was noted by Bjerrum, Simons and Torblaa (1958), who considered that the phenomenon might be analogous to secondary consolidation, though the possibility of osmosis between the pore water and the water in the cell had to be considered.

The significance to be attached to stress paths (and pore pressure parameters) based on different rates of testing and conditions of end restraint, particularly in undisturbed samples prone to zone failure, deserves further discussion.

UNDRAINED TESTS ON PARTLY SATURATED SAMPLES

The importance of discriminating between and measuring separately the pore water and pore air pressures in partly saturated soils has been the subject of a number of papers (Bishop (1960), Gibbs et al (1960), Barden and Sides (1967), Barden, Madedor and Sides (1969)). The difficulties are most acute where:

a) the air voids are discontinuous and the equalization of air pressure is very slow. This only occurs at relatively high degrees of saturation.

b) the degree of saturation is very low and the equalization of pore water pressure is very slow.

c) the difference between the pore air and pore water pressure exceeds the air entry value of the finest grain size ceramic available.

d) the initial pore water pressure is

below zero absolute, which leads to cavitation in conventional hydraulic measuring systems.

In case a) the presence of air is significant mainly in its effect on the compressibility of the pore fluid. Its pressure plays little part in the effective stress equation. The fact that the air pressure measurement presents difficulties is thus of little practical importance.

In case b) the difficulty can be met to some extent by the use of small samples and low testing rates. Case c) will generally occur only if the sample falls into category d) as well. In both cases a complete picture can only be obtained by inferences from indirect tests or tests involving either an increase in pore air pressure or in total stress. Examples of such procedures are given by Bishop et al (1960 a,b), Croney and Coleman (1960).

CONSOLIDATION TESTS AND PORE PRESSURE DISSIPATION TESTS

The determination of the rate of dissipation of pore pressure in the consolidation test is both of great theoretical interest (for example, Taylor (1942), Leonards and Girault (1961) Bishop and Al-Dhahir (1969)) and also of considerable practical importance, in particular in partly saturated soils. The determination may be made in the standard consolidation test (for example, Whitman, Healy and Richardson, 1961), in the triaxial apparatus (for example Bishop and Henkel, 1962) or the hydraulic oedometer (for example, Lowe, Zaccheo and Feldman, 1964: Rowe and Barden, 1964 a; Raymond, 1966). The latter two types of test have certain advantages over the conventional oedometer in that:-

a) the undrained pore pressure may be determined with a sealed system before drainage is allowed to commence,

b) a back pressure may be used to ensure full saturation where this is appropriate,

c) a series of consolidation stages may be carried out by the successive reduction of the back pressure in the drainage system without any change in the total stress applied to the sample.

The work of Whitman et al (1961) clearly shows the importance of matching the transducer and filter characteristics to the coefficient of consolidation and drainage path of the sample. In this connection, it is relevant to note that the permeability and dimension of the drainage surface may also have a marked effect on the apparent coefficient of consolidation if incorrectly chosen relative to those of the sample, Newland and Allely (1960), Bishop and Gibson (1963).

MODEL TESTS

Laboratory measurements of pore pressure on models fall into two main categories:

a) Measurements of pore pressure set up during steady seepage in relatively pervious cohesionless soils.

b) Measurements of transient pore pressure set up by stress changes in soils where permeability is low relative to the rate of loading. Model footing and model wall tests fall into this category.

The apparatus for steady seepage tests is simple (for example, Reinius, 1948), but as the pressures are very low, care has to be taken over capillarity, trapped air bubbles, etc.

Few transient pore pressure results have been reported. Burland and Roscoe (1969) describe an apparatus designed for this purpose, but give results for one dimensional consolidation only. Problems of installation, sealing and the avoidance of trapped air are considerable. A minimum time lag is particularly desirable due to the high pore pressure gradients in small models, and a high sensitivity is necessary since the pore pressure changes are small.

CONCLUSIONS

Techniques are available which permit the measurement of pore pressure in undrained triaxial tests, in consolidation tests and in model tests with increasing accuracy and convenience. However, continual vigilance is necessary to ensure that the measured pore pressure is not influenced by the testing technique or the physical characteristics of the measuring equipment.

Discussion by Professor Anwar E. Z. Wissa (Massachusetts Institute of Technology, USA)

In his opening statement on the laboratory measurement of pore pressures, Professor Bishop stated that progressive equalization of non-uniformity in pore pressure within a laboratory test specimen, resulting from end restraint or from a natural tendency to zone failure, is a much more serious cause of erroneous pore pressure measurements than the volume factor (compressibility) of the pore pressure measuring system. I agree with Professor Bishop's statement for the specific case that occurs in undrained triaxial tests when the pore pressure is measured at the base of the test specimen during shear. I should like to mention that the compressibility of the pore pressure measuring system can also introduce very large errors that are time independent. These errors can be especially large when testing stiff soils such as shales, heavily overconsolidated clays, cemented soils and dense sands at high consolidation pressures which have a compressibility in the order of

10^{-4} cm^2/kg.

Further, when pore-water pressures are measured in one-dimensional consolidation tests (for example, in the constant rate of strain consolidation test, I described in Specialty Session 16) and during isotropic consolidation and pore pressure dissipation tests, non-uniformity of excess pore-water pressure does not occur and then the primary inherent source of error is due to the compressibility of the pore pressure measuring system.

The pore pressure response, B, of a laboratory test set-up is given by the following equation:

$$B = \frac{\Delta u}{\Delta \sigma} = \frac{1}{\frac{1}{B_{theo.}} + \frac{1}{V_o C_{sk}} (V_L C_W + C_L + C_M)}$$

where

- B is the undrained pore pressure response and is equal to the measured increment of pore-water pressure, Δu, caused by an applied increment of total stress, $\Delta \sigma$.
- B_{theo} is the theoretical pore pressure response of the test specimen excluding the influence of the pore pressure system and is by Skempton's equation. $B_{theo} = 1/(1 + n\ C_W/C_{sk})$ where n is the porosity, C_W the compressibility of the pore fluid and C_{sk} the compressibility of the mineral skeleton.
- V_o is the volume of the test specimen.
- V_L is the volume of fluid in the pore pressure measuring system.
- C_L is the compressibility of the pore water lines.
- C_M is the compressibility of the pore pressure sensing element.

From the above equation it is seen that the error in measuring the pore pressure can be reduced by either increasing the volume of the test specimen or by reducing the compressibility of the pore pressure measuring system. The possibility of increasing the volume of the test specimen is usually limited by the dimensions of the available undisturbed bore-hole sample which rarely exceeds four inches in diameter. In addition, with the exception of the pressure measuring system the cost of laboratory testing equipment increases rapidly with increasing size of the test specimen. Finally, increasing the size of the test specimen decreases the rate of consolidation and pore pressure dissipation or equilization. For these reasons it is usually impractical to improve the pore pressure response by increasing the volume of the test specimen.

For a properly designed pore-water pressure measuring system which is completely deaired $C_L \to 0$ and $C_W \approx 5.5 \times 10^{-5}$ cm^2/kg, i.e. the compressibility of deaired water. While V_L of most systems currently in use is at least 12 cc, existing testing equipment can easily be modified to reduce the volume of water in the pore pressure measuring system to about 1 cc. Further, by using a stiff electrical pressure transducer rather than a null indicator as the pressure sensor it is possible to reduce C_M from about 1.4×10^{-3} to 1.6×10^{-5} cc/kg/cm^2. By making these two modifications to the pore pressure measuring system the error in reading equilibrium pore pressure of saturated soils can be reduced from about 17% to less than 1% for a stiff test specimen having a compressibility of 10^{-4} cm^2/sec, a porosity of 0.5 and a volume of 80 cc. It should be mentioned that for more compressible soils with C_M greater than 10^{-3} cm^2/sec these modifications are not needed since the error in measuring the pore pressure is then less than 2-1/2%

These problems are also discussed by Wissa (1969).

<u>Discussion by Mr. H. J. Gibbs</u> (U.S. Bureau of Reclamation)

In the introductory notes of Bishop and Green the variability of pore pressure in the test specimen was emphasized. The Bureau of Reclamation studies suggest that the equalization of pore pressure throughout the specimen is relatively rapid when the specimen is compressed under sealed (undrained) conditions, since a major transfer of the pore fluid does not occur. Greatest emphasis in these studies was given to the improvement of accuracy in the test measurement, such as preventing fluid transfer between the pore pressure contact and the soil, making the system truly sealed so that rapid response can be realized. Techniques in measurement that were described are:

(1) Pore-air pressure contacts in which the water films of the coarse ceramic is separated from that of the soil and there is maintained a precise control of the water level in the ceramic contact.

(2) Precise measurements of initial capillary pressure and its change as compression occurs permit theoretical curves of pore pressure to be established. These are used to guide the

rate of testing so that reasonable equalization of pore pressure is realized when measured values fall along them.

Further details of these techniques are given by Gibbs and Coffey (1969).

Discussion by Professor K. V. Helenelund (Institute of Technology, Otaniemi, Finland)

Professor Helenelund discussed the laboratory measurement of pore pressure in fibrous soils and in particular the laboratory consolidation of peat. A system was described using two oedometers and four pore pressure probes in each oedometer.

The results indicated important differences between the horizontal and vertical permeability of peat.

In unconfined compression tests on samples of peat cut with their axis vertical the increase in pore pressure during loading was high. However, in samples cut with their axis horizontal the increase in pore pressure was small or zero.

The results also raised the important question of the relevance of the pore pressure within the fibre relative to that within the pore space.

Written discussion by Professor Anwar E. Z. Wissa (Massachusetts Institute of Technology, U.S.A.).

At the specialty session Professor Bishop discussed the influence of progressive equalization of non-uniformity in pore pressure in laboratory measurements of pore-water pressure and the writer discussed the effects of compressibility (volume factor) of the pore pressure measuring system. Other sources of error in the laboratory measurement of pore-water pressures were mentioned in the introductory notes, but were not discussed at the session and therefore it may be useful to briefly describe the experimental procedures used at M.I.T. to detect these errors. The most common additional causes for erroneous pore-water pressure measurements are:- incomplete saturation, external and internal leakage.

Figure 3 shows a typical triaxial test setup in which the pore-water pressure is measured with a mercury null indicator. Starting with all valves shown in the figure closed, the sample is first consolidated under the desired back pressure by applying the cell pressure at the same time as the back pressure through Valves A and B. When consolidation and saturation are complete, as determined by the volume change burette, valves A and B are closed and the system is checked prior to undrained shear as follows:

1. Leakage checks

The consolidation back pressure is applied to the pore pressure measuring system (left hand side of Fig. 3) by opening Valves D, E, F, G and H. Once the back pressure has built up in the system it is applied to the test specimen by opening Valve C. The mercury level in the null indicator is zeroed with screw J and the null indicator by-pass Valve D is closed. The mercury level in the left column of the null indicator is then monitored as a function of time. If consolidation and saturation of the test specimen are complete and there are no leaks the mercury level remains stationary. A rise in the mercury level as a function of time can be due to several reasons:

(a) Air still going into solution due to incomplete saturation of the sample or pore water lines.
(b) Incomplete swelling of the sample if it is being rebounded.
(c) External leakage at Valves B or C or in the lines connecting the cell base to the null indicator.

If the cause is reason (a) or (b) above then the rate of rise of the mercury level in the null indicator will be decreasing with time and the sample should be allowed to further equilibrate before shearing. A constant rate of rise of the mercury level indicates an external leak which must be eliminated before starting undrained shear and is usually too small to visually detect.

A drop in the mercury level as a function of time is caused by either internal leakage or incomplete consolidation. Incomplete consolidation will be reflected by a decrease in the rate of drop of the mercury level as a function of time and the sample must be allowed to further equilibrate before testing. Internal leakage due to flow of cell water into the test specimen results from poor sealing or a tear in the membrane and shows up as a constant rate of drop of the mercury level with time. If this is the case the sample must be taken down.

When an electrical pressure transducer is used instead of a null indicator to measure pore-water pressure, the system on the right hand side of Valve C in Fig. 3 is eliminated and the transducer in its mounting block is connected directly to the cell pore-water port in the place of Valve C. Following consolidation and saturation, Valve B is slowly closed and the pore-water pressure is then monitored as a function of time with the transducer. In this case Valve B must not cause a volume change during opening and closing since this would induce a pore-water pressure change in the sample. An external leak shows up as an approximately constant rate of pore pressure drop with time whereas an internal leak causes an approximately constant rate of increase in pore pressure with time. Incomplete saturation or rebound causes a drop in pore pressure; the rate of which decreases with time whereas incomplete consolidation causes the pore pressure to increase at a decreasing rate.

Saturation Check

Following the leakage checks the pore pressure response of the system is checked to make sure the sample and pore pressure measuring system are completely saturated. With Valve B closed the cell pressure is gradually increased by a known amount and the corresponding induced pore-water pressure is measured with the null indicator, test gauge and screw pump or by the pressure transducer. If the measured increase in pore pressure, Δu, is equal to the applied increase in cell pressure, $\Delta\sigma$

$B = \frac{\Delta u}{\Delta \sigma}$ is unity and

the system and test specimen are completely saturated. An increase in pore water pressure less than the applied increase in cell pressure ($B<1.0$) is either due to air still remaining in the sample and/or pore water lines or due to the stiffness of the test specimen. If this is the case the cell pressure and back pressure are simultaneously increased by an equal amount following which another pore pressure response is taken. If the new value of B is larger than the first value obtained at the lower back pressure but still less than unity then the system is not yet completely saturated and the pore-water lines should be flushed and the sample allowed to equilibrate at this higher back pressure before rechecking for leakage and saturation. If the new value of B is still less than unity but equal to the value obtained at the lower back pressure the system is completely saturated and testing can proceed.

In summary, based on the experiences the writer has had with pore pressure measurements in the laboratory, it is his opinion that reliable measurements can only be obtained if a systematic checking procedure such as that presented above is undertaken prior to each test. Further, electrical pressure transducers, when properly calibrated and checked are simpler and more accurate than null indicators for pore-water pressure measurements in the laboratory.

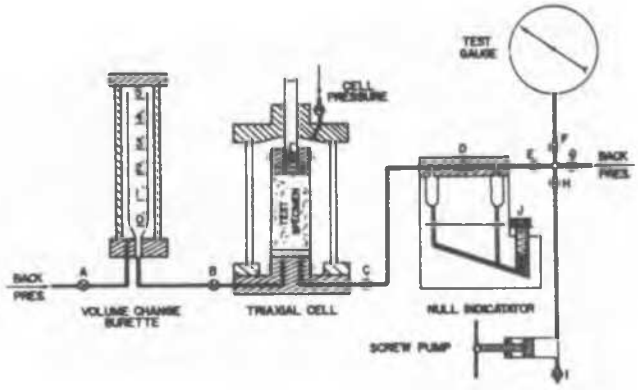

Figure 3

TRIAXIAL TEST SETUP FOR PORE WATER PRESSURE MEASUREMENTS
(A. E. Z. WISSA)

References

ARHIPPAINEN, E. (1964) Pore pressure measurements in two Finnish earth dams. 8th Cong. Large Dams, II; 503.

BARDEN, L., A. O. MADEDOR and G. R. SIDES (1969) Volume change characteristics of unsaturated clay. Jour. Soil Mech. Div., ASCE, Vol. 95, No. SM 1, 33-51.

BARDEN, L. and R. J. W. McDERMOTT (1965) Use of free ends in triaxial testing of clays. Jour. Soil Mech and Found. Div. ASCE, Vol. 91, No. SM 6, 1-23.

BARDEN, L. and G. R. SIDES (1967) The diffusion of air through the pore water of soils. Proc. 3rd Asian Reg. Conf. on Soil Mech. and Found. Eng., Haifa, 135-138.

BARGE, J., G. POST and P. HUYNH (1964) Auscultation de la Digue Serre-Poncon. 8th Cong. Large Dams, II; 351.

BAZETT, D. J. (1970) The characteristics of till placed under water in the Arrow Dam. 10th Int. Cong. Large Dams (to be published).

BISHOP, A. W. (1960) The measurement of pore pressure in the triaxial test. Proc. Pore Pressure and Suction in Soils Conf., London, 38-46.

BISHOP, A.W. and Z. A. AL-DHAHIR (1969) "Some comparisons between laboratory tests, in situ tests and full scale performance, with special reference to permeability and coefficient of consolidation". Proc. Conf. on 'in situ investigations in soils and rocks;' 161-174. British Geotechnical Society.

BISHOP, A. W., I. ALPAN, G. E. BLIGHT and I. B. DONALD (1960 a) Factors controlling the strength of partly saturated cohesive soils. Proc. Res. Conf. on Shear Strength of Cohesive Soils, Boulder, ASCE, 503-532.

BISHOP, A. W., I. ALPAN, G. E. BLIGHT and I. B. DONALD (1960 b) Closure of "Factors controlling the strength of partly saturated cohesive soils". Proc. Res. Conf. on Shear Strength of Cohesive Soils. Boulder, ASCE, 1027-1042.

BISHOP, A. W. and A. K. G. ELDIN (1950) Undrained triaxial tests on saturated sands and their significance in the general theory of shear strength. Geotechnique, 2; 13-22.

BISHOP, A. W. and R. E. GIBSON (1963) The influence of the provisions for boundary drainage on strength and consolidation characteristics of soil measured in the triaxial apparatus. Symp. Laboratory Shear Testing of Soils, Ottawa, ASTM, STP 361, 435-451.

BISHOP, A. W. and D. J. HENKEL (1962) The measurement of soil properties in the triaxial test. Arnold, London, 2nd Edit.

BISHOP, A. W., M. F. KENNARD and P. R. VAUGHAN (1964) Developments in the measurement and interpretation of pore pressure in earth dams. 8th Cong. Large Dams, I; 47.

BJERRUM, L., N. E. SIMONS and I. TORBLAA (1958) The effect of time on the shear strength of a soft marine clay. Proc. Brussels Conf. on Earth Pressure Problems, Vol. 1. 148-158.

BLIGHT, G. E. (1961) Strength and consolidation characteristics of compacted soils. Ph.D. Thesis, University of London.

BLIGHT, G. E. (1963 a) The effect of non-uniform pore pressures on laboratory measurements of the shear strength of soils. Symp. Laboratory Shear Testing of Soils, Ottawa, ASTM, STP, 173-184.

BLIGHT, G. E. (1963 b) Discussion on "Pore pressures within soil specimens in triaxial compression". Symp. Laboratory Shear Testing of Soils, ASTM, STP 361, 199-204.

BLIGHT, G. E. (1965) Shear stress and pore pressure in triaxial testing. Jour. Soil Mech. and Found. Div., ASCE Nov. 1965, 25-39.

BOLT, G. H. and J. V. LAGERWERFF (1965) Consequences of electrolyte redistribution during pressure membrane equilibration of clays. Soil Science, 99:3:147.

BROOKER, E. W., J. S. SCOTT and PHYSOOL ALI (1968) A transducer piezometer for clay shale. Canadian Geotech. Jour., V (4); 256.

BURLAND, J. B. and K. H. ROSCOE (1969) Local strains and pore pressures in a normally consolidated clay layer during one-dimensional consolidation. Geotechnique, 19:3:335-356.

CASAGRANDE, A., H. Q. GOLDER and D. J. BAZETT (1969) Design and testing of the Arrow Dam. Presented to ASCE Water Resources Conf. Feb. 1969, (to be published)

CIVIL ENGINEERING (1967) Piezometers monitor waterfront fill. Civ. Eng. ASCE. June.

CONTE, J. and P. R. CHANEZ (1964) The Dam of Notre-Dame. Travaux, April.

COOLING, L. F. (1962) Field measurements in Soil Mechanics. Geotechnique, 12(2); 75.

CRONEY, D. and J. D. COLMAN (1960) Pore pressure and suction in soil. Proc. Pore Pressure and Suction in Soils Conf., London, 31-37.

DAEHN, W. W. (1962) Development and installation of piezometers for the measurement of pore fluid pressures in earth dams. ASTM Symp. Soils and Found. Eng.

ESCARIO, V. (1969) A new method for insitu measurement of pore water tension. Int. Conf. on Expansive Soils, College Station, Texas.

GIBBS, H. J. and C. T. COFFEY (1969) Techniques for pore pressure measurements and shear testing of soil. Proc. 7th Int. Conf. on Soil Mechs. and Found. Eng. Vol. 1; 151-157.

GIBBS, H. T., J. W. HILF, W. G. HOLTZ and F. C. WALKER (1960) Shear strength of cohesive soils. Proc. Res. Conf. on Strength of Cohesive Soils, ASCE, 33-162.

GIBSON, R. E. (1963) An analysis of system flexibility and its effect on time-lag in pore-water pressure measurements. Geotechnique, 13 (1); 1-11.

GOLDER, H. Q. and D. J. BAZETT (1967) An earth dam built by dumping through water. 9th Int. Cong. Large Dams, Vol 6; 369.

HALL, E. B. and B. B. GORDON (1963) Triaxial testing with large scale high pressure equipment. Symp. Laboratory Shear Testing of Soils, ASTM, STP 361, 315-328.

HENKEL, D. J. and V. SOWA (1963) Discussion on "Creep studies on saturated clays". Symp. Laboratory Shear Testing of Soils. ASTM, STP 361, 104-107.

HILF, J. W. (1956) An investigation of pore water pressure in compacted cohesive soils. Tech. Mem. 654, U.S.B.R., Denver, Colorado.

JOHNSON, R. W. and E. J. YODER (1963) Repeated-load triaxial test of soils with pore pressure measurement. ASTM, Proc., Vol. 63, 1207-1226.

JUAREZ-BADILLO, E. (1969) Failure theory for clays. Proc. 7th Int. Conf. Soil Mechs. & Found. Eng., Vol. I, 203.

LAUFFER, M and W. SCHOBER (1969) The Gespatch Rockfill Dam in the Kauner Valley. Trans. 8th Cong. Large Dams, III; 635.

LEONARDS, G. A. and P. GIRAULT (1961) A study of the one-dimensional consolidation test. Proc. 5th Int. Conf., Soil Mech., Vol I, 213-218.

LITTLE, A. L. (1964) Discussion, 8th Cong. Large Dams, V; 292.

LOWE, J., P. F. ZACCHEO and H. S. FELDMAN (1964) Consolidation testing with back pressure. Jour. Soil Mech. Found. Div. ASCE, Vol. 90, SM 5, 69-86.

LUNDGREN, R. (1966) Electric piezometer for field installation. Civ. Eng. ASCE, August.

MORGAN J. R. and P. J. MOORE (1968) Chapter 5, "Experimental techniques", in Soil Mechs. - Selected Topics, Edit. by I. K. Lee, Butterworths.

McLAREN, D. (1968) M.6 Trial Embankment at Killinton. Road Res. Lab., Report LR238.

NASH, K. L. and R. K. DIXON (1960) The Measurement of pore pressure in sand under rapid triaxial test. Proc. Pore pressure and Suction in Soils Conf., London, 21-25.

NEWLAND, P. L. and B. H. ALLELY (1960) A Study of the consolidation characteristics of a clay. Geotechnique, 10; 62-74.

OLSON, R. E. (1960) The migration of pore fluid during shear in consolidated undrained (R) triaxial tests. Proc. ASCE Res. Conf. on Shear Strength, June, 1943.

PENMAN, A. D. M. (1956) A field piezometer apparatus. Geotechnique, 6:2:57.

PENMAN, A. D. M. (1960) A study of the reponse times of various types of piezometers. Proc. Conf. Pore Pressure and Suction in Soils, Butterworths, London.

PETERS, J. D. (1963) A gauge for measuring pore water pressures in soil under dynamic loads. Ph.D. Thesis, U. of Illinois, 76 pp. (Univ. Micr. Serv. 64-2941)

PINKERTON, I. L. and A. C. McCONNELL (1964) Behaviour of Tooma Dam. 8th Cong. Large Dams, II; 351.

RAYMOND, G. P. (1966) Consolidation of slightly overconsolidated soils. Jour. Soil Mech. and Found. Div., ASCE., Vol. 92 SM 5, 1-20.

REINIUS, E. (1948) The stability of the upstream slope of earth dams. Meddelanden Statens Kommitte for Byggnadsforskning, Stockholm. No. 12.

RICHARDS, B. C. (1969) Psychrometric techniques for measuring soil water potential. C.S.I.R.O., Div. Soil. Mech. Tech. Rep. No. 9.

ROWE, P. W. (1968) The influence of geological features of clay deposits on the design and performance of sand drains. Proc. I.C.E., Supplementary Volume.

ROWE, P. W. and L. BARDEN (1964 a) The importance of free ends in triaxial testing. Jour. Soil Mech. and Found. Div., ASCE Jan.

ROWE, P. W. and L. BARDEN (1964 b) A new consolidation cell. Geotechnique, 16 (2); 162-170.

SCHOFIELD, A. N. and C. P. WROTH (1968) Critical State Soil Mechanics, McGraw-Hill, London, 168 pp.

SCOTT, J. D. and J. KILGOUR (1967) Experience with some vibrating wire instruments. Canadian Geotech. Jour. IV (I); 100.

SHEPPARD, G.A.R. and A. L. LITTLE (1955) Stabilishing an earth foundation by means of sand drains. 5th Int. Conf. Large Dams, Vol. 1, P. 639.

TAYLOR, D. W. (1942) Research on consolidation clays.
MIT Report No. 82.

TAYLOR, D. W. (1944) Tenth progress report on shear research to U.S. engineers.
MIT publication.

TORBLAA, I. (1966) Poretrykksmaler utfort ved Dam Hyttejuvet med fonskjellig maleutstyr.
N. G. I., 68; 25.

TSCHEBOTARIOFF, G. P., E. R. WARD, E. DIBIAGIO and J. WATKINS (1956) Large scale triaxial cell tests.
Progress report to Office of Naval Research, Princeton Univ., N.J.

VAUGHAN, P. R. (1965) Field measurements in earth dams.
Ph.D. Thesis, Univ. of London.

VAUGHAN, P. R. (1969) A note on sealing piezometers in boreholes.
Geotechnique, 19:3:405

WARLAM, A. A. and E. W. THOMAS (1965) Measurement of hydrostatic uplift pressure on spillway weir with air piezometer.
ASTM, STP, 392; 143,

WHITMAN, R. V., K. A. HEALY and A. RICHARDSON (1961) Time lags in pore pressure measurements.
Proc. 5th Int. Conf., Vol. 1, 407-411.

WISSA, E. Z. (1969) Pore pressure measurement in saturated stiff soils.
Proc. ASCE, Jour. Soil Mechs. and Found. Div., Vol 95, No. SM4, 1063-1073.