# TECHNICAL SESSION No. 8—MEASUREMENT TECHNIQUES AND INSTRUMENTATION

"An Investigation of an Earth Pressure Problem using a Rod Model Analogue", by R. Butterfield & K.Z. Andrawes.

"Instrumentation of Raft Foundations in Perth", by C.M. Gerrard, M. Kurzeme, D.C. Andrews & R. Topp.

"An Infinitely Programmable Stiff Loading Frame", by O.G. Ingles & R.C. Neil.

"Patterns of Strain in Strength Test Samples", by W.M. Kirkpatrick & J.S. Younger.

"Simple Cantilever Beam Instrumentation for the Determination of Creep Behaviour in Rocks", by L.M.L. Klingmueller & M.J. Wallace.

"Psychrometric Techniques for Field Measurement of Negative Pore Pressure in Soils", by B.G. Richards.

"Laboratory Shear Testing of Weakness Planes in Diamond Drill Core", by E.P. Waghorne.

GENERAL REPORTER - Mr. J.M. MADDOX:

Introduction:

The title Measurement Techniques and Instrumentation casts a net over a large part of the science and art of Geomechanics. Virtually any study in geomechanics contains results of some measurements made somewhere, either in experimental testing procedures, or in model analyses of particular problems, or in investigation of the behaviour of actual structures or excavations. No doubt the criterion used by the Conference Committee in allocating papers to this session was whether the particular measurement technique discussed was itself the most important aspect of the paper. Improved measurement of relevant properties is certainly the theme of all seven papers in this session. There is some sense of regret that this subject did not draw a greater response and coverage, because one feels that the greatest possibilities for progress in geomechanics today are in improved and more economical measuring techniques.

It is appropriate to consider the economics of measurements and instrumentation, because the costs of investigations are often quite formidable. Two of the papers are directly concerned with more economical testing, while a third recognises the relatively high costs of a particular in-situ investigation. In general terms it is not sufficient to estimate the cost of an investigation and justify that cost simply as a percentage of the total for the project. A real value analysis should be required before any investigation is undertaken. While such a proposition might be rejected or disregarded by those engaged in research and development of techniques, these researchers are often more conscious of costs than those who are applying the techniques, or those who are requisitioning actual investigations. This is due primarily to lack of appreciation of the difficulty of making real and meaningful measurements.

Almost any measurement is more difficult than is generally supposed, and in geomechanics the problems are even greater than usual. This difficulty is of the greatest importance because making meaningful measurements of soil or rock parameters which are appropriate for any particular situation, and assessing the precision of those measurements in relation to the factors of safety to be applied in design is what practical geomechanics is. An interesting paradox in civil engineering is the low factor of safety frequently used for earth structures, where the lack of precision in measuring true and representative parameters is notable, compared with those used for concrete or steel structures, for which the uncertainties are considerably less. The deceptively simple concepts of shearing resistance, deformability and permeability, when applied to geomechanics, provide many interesting problems in determining parameters which are even approximately correct for a given set of conditions. There are always uncertainties in any geomechanics analysis, and sometimes assumptions are so fundamental that the evaluation is at best only a rough approximation. It is the responsibility of those who have made, or have directed the measurement of physical properties to ensure that the limitations of the information available are fully appreciated and that the results are applied with proper caution.

There are, in fact, two problems: knowing what to measure, then knowing how to measure it with sufficient accuracy. The first of these is beyond the scope of this session, but one example will illustrate the point. In investigations of rock mechanics problems many engineers order unconfined compressive strength tests of the rock material because it is the most obvious property to measure. Yet in a large proportion of rock mechanics problems the compressive strength is of little value (Obert, 1971). However, there are difficulties enough for this discussion in the matter of making correct measurements with sufficient accuracy.

The first task in any investigation is usually identification and description of the material, and here the engineer deliberately does this in terms of form rather than substance. He is more concerned with physical state than mineral composition; plasticity of clay; particle size distribution of granular material; and spacing and nature of discontinuities in rock masses. Even at this early stage errors may arise. Particle size distribution (Turnbull, 1970) or plasticity tests if not performed on appropriately treated samples can be seriously in error. One of the papers in this session is concerned with the considerable difficulty of making relevant soil moisture measurements. Inadequate exploration of rock masses, and inadequate information about the structural geology, or misinterpretation of the information available often leads to poor identification.

After identification the problems of measurement are compounded, and each determination may further misrepresent the true state of affairs. For instance, specific gravity values vary according to the testing procedure used, and density of granular materials is remarkable for inconsistency, so evaluation of voids ratio may be in error from both sources. Anyone who has attempted to determine maximum and minimum densities in the evaluation of relative density is aware that results vary with different operators and with different equipment. In-situ density is also difficult to reproduce, and the larger the maximum particle size the more widely the results vary.

So it is with considerable caution that one approaches the task of measuring shear strength, compressibility and permeability, and it is with equal caution that one views the results. The possible variation in results is well illustrated in the determination of the coefficient of friction between two rock surfaces. What is often thought of as a single, simple relationship between normal and shearing forces is actually a variable ratio depending upon the roughness and the unevenness of the particular specimens being tested, the presence of moisture, the presence of discrete particles between the surfaces, the history of previous movement, the intensity of pressure, and the manner of movement permitted in the test. For example, Horn and Deere (1962) showed that moisture can have a very large effect upon the frictional coefficients of very smooth rock surfaces. The static coefficient of friction for a massive-structured mineral (microcline feldspar) was 0.12 when dry, but 0.77 when wet, while the static coefficient for a layer-lattice mineral (chlorite) was 0.53 when oven dry, and 0.22 when saturated. Fortunately the effect of moisture is considerably less than this for natural surfaces, but, on the other hand, the roughness, unevenness, and lack of planarity of natural surfaces result in even greater differences in resistance to shear.

In summary one can say that no physical property in geomechanics has anything like precise values, and no measurement is exact. For this reason contributions to the science which improve the accuracy, relevance or meaning of measurements are particularly welcome.

Introduction of Papers:

Measurement techniques may be conveniently divided into three groups:

(a) laboratory testing;

(b) models; and

(c) in-situ instrumentation of structures and foundations.

Each of these groups is represented in the seven papers presented.

Laboratory Testing:

Interest has been growing in recent years in equipment which can load specimens of brittle material with strain fully controlled. To allow strain to continue at the predetermined rate as rupture approaches, the machine used must be capable of shedding excess load very rapidly, and this excess includes the reaction energy stored in the machine. Ingles and Neil describe instrumentation in association with a standard loading frame and a small computer with sufficiently rapid response to allow full control over strain at rupture of soft rocks and coal. The system is reported to be capable of off-loading stored energy represented by a frame deflection of 0.05 cm. at 30 tons load in 0.002 sec., which would seem to be a significant achievement. The authors claim that this system permits a greater range of control programme than commercially available servo-control loading frames.

A little more information concerning strain measurement would be of interest, particularly the frictionless band technique to give mean radial strain, and the proposed volumetric strain device. One assumes that all tests in which this equipment is used would be on specimens with "frictionless" platens to ensure uniform radial strain. An alternative method for measuring volumetric strain over the central section of specimens developed in the H.E.C. Tasmania laboratories is described in a discussion of this paper. Many engineers in the geomechanics field will appreciate the availability of such a sophisticated testing machine for controlled strain tests, as the cost of setting up a similar one would not be small.

Klingmueller and Wallace have devised a "simple, inexpensive, reliable and accurate system" for bending creep tests on specimens of rock material. The adjectives are appealing, as they must have been to the financial supporters of the study. The only matter which may affect the simplicity and cheapness of the apparatus is the need for humidity control, but perhaps close control is not required for most work and therefore easily obtained. The method measures the deflection of very small (6 cm. x 1.5 cm. x 0.3 cm.) cantilevered beams of rock. Increments of load are applied with lead shot and deflections are measured by micrometer with an electrical contact sensor.

The arrangement is attractive because, apart from its simplicity, the very small specimens would allow multiple tests on essentially the same material. The creep characteristics of each piece of material can then be amply studied at low cost and analysed statistically. With a high degree of confidence in the parameters of any one sample of rock, full attention can then be given to variations in the material itself. Having said this, one wonders whether there is really much to be gained from testing many specimens from one sample of rock, if the variability between a set of specimens is in fact small compared with the variability between different samples.

The most readily available samples of rock for testing are usually diamond drill cores, and are thus preferred, from the point of view of low cost, to other samples for laboratory testing. In any case drill coring is often the only means of sampling rock. Various arrangements have been made for measuring the frictional resistance of joints in rock core. Waghorne has described a direct shear apparatus for testing joints in rock core which form an angle with the core axis greater than 30°. The principal advantage of the apparatus is that it permits rapid setting up of core for test with the joint oriented in the plane of shearing and rotated to the required direction of shear. The alternative to this mechanical setting is to embed each specimen of jointed core at the required orientation in plaster or concrete - a slower and therefore more costly process.

Because most joints are not particularly plane, drill core specimens are sometimes considered to be too small to allow a representative area of the joint to be tested. This is generally true, but then almost any area of joint which can reasonably be isolated for testing is too small to be representative. The best that can be done is to measure the frictional resistance of a limited area of joint, observing any movement normal to the plane which occurs, then to "interpret" the result with regard to whatever larger scale undulations and discontinuities that can be observed in the joint. This interpretation is frequently much more difficult than testing representative specimens for shearing resistance. Allen (1971) in a paper to this Conference describes such a study and Hoek (1971) expresses the same views about shear testing of joints.

However there are still difficulties with testing procedures in relation to the natural conditions. A testing apparatus such as that described allows shear in only one plane and one fixed direction, whereas on the actual joint some freedom in direction of movement, twisting and tilting, as well as a degree of progressive failure may be possible, leading to a reduced coefficient value. While the author may conclude "...that this machine has fulfilled the requirements of providing measurements of peak and residual frictional properties of joints", it is suggested that such measurements are only starting points in determining shearing resistance of joints.

Models:

Butterfield and Andrawes have revived and refined the rod model analogue for investigating earth pressure problems. The principal refinement is the use of stereo-photogrammetry to plot contours of displacement of the rods, a much less tedious process than measuring displacements by hand on the model or on photographs. Use of a survey plotting machine allows the contours to be drawn in quite a short time with much greater precision. Displacement measurements of the order of 0.01mm. are possible on the photographic plates used for plotting. The technique is described in detail in a previous paper by the authors (1970), but consists essentially of taking photographs of movement normal to the axis of the lens of a fixed camera and viewing as stereoscopic pairs, in the same way as conventional stereo-photogrammetry involving photographs of a stationary object taken by moving camera. The section of the model which has moved appears to be elevated, the elevation, which may be scaled, being proportional to the displacement. The particular model investigation described in deformation "at low stress levels behind a fairly smooth rigid model retaining wall rotating about its top into the fill". Comparison is made with results from a similar model using sand backfill. The retaining wall itself consisted of seven full-width load cells. The authors show that wall pressures developed in the rod model are similar to those in loose sand, but the kinematic behaviour is quite different from that in loose sand. Kinematic behaviour in the rod material more closely reproduces that in a densely compacted sand.

The General Reporter can only comment that a rod model analogue of this type was made by H.E.C. Tasmania some years ago, following communication between Mr. T. Farrant of Adelaide University and Mr. J.K.Wilkins, but was not pursued because of the difficulty of measuring displacements accurately. The authors' method would seem to overcome this difficulty.

With a view to more accurate and more meaningful testing Kirkpatrick and Younger have been examining patterns of strain in strength test specimens. In an earlier paper (1970) they showed, working with single size, dry, medium density sand, that uniform axial strain is achieved in compression tests only when lubricated platens are used. Earlier still, Kirkpatrick and Belshaw (1968) showed how lateral strain distribution is improved with lubricated platens. This paper extends the work to include extension tests on cylindrical specimens and extension and compression tests on cubical specimens. All of these tests used single size Leighton Buzzard sand at medium density, and the displacements were measured from radiographs of the specimens which contained grids of lead shot. These papers have quantified what has been tacitly recognised for some time. The authors, perhaps cautiously, state in conclusion that "rough platens should be avoided where reliable stress - strain information is required", i.e., when estimation of strains from displacements measured at specimen boundaries are required.

The question arises as to whether standard procedures for compression tests should be changed, as those in use at present depart markedly from the ideal of uniform axial strain over a known volume of specimen. Should all strength tests be carried out with lubricated platens? Tests carried out in the H.E.C. laboratories on standard concrete specimens, of 2:1 ratio of height to diameter, have shown that removal of platen restraint reduces "strength" by 20-25%. The mode of failure is changed from shear to tension (splitting) and may be more appropriate for many applications.

In-Situ Instrumentation:

Instrumentation of soils and rocks in civil engineering works is quite common, particularly in earth and rockfill dams. However instrumentation of building foundations, beyond survey measurements, seems to be rare - and probably difficult to justify in most cases. Gerrard, Kurzeme, Andrews and Topp describe the instrumentation of soft foundations of three new bank buildings in Perth, which must surely be a record in comprehensive and detailed study of the behaviour of such foundations in Australia. How fortunate to find proprietors of three multi-storey building projects on neighbouring sites willing to contribute to the cost of such a study. Instruments were installed to measure

contact stresses under rafts,
pore pressures under rafts and at depth,
total settlement and deflected shape of rafts,
settlement at depth,
lateral movement at depth,
stresses and strains within rafts, and
column loads.

The authors note that "relative to usual investigation costs, field measurement projects are extremely expensive to undertake, and it is important to examine their probable economic returns." This would seem to be easier said than done, and some idea of the methods of estimating economic returns would be of interest. It is worth mentioning here some of the cost factors involved in instrumentation, because the total cost of an installation can be severely under-estimated by neglecting some of the following:

capital cost of sensing instruments,
" " " reading instrument,
" " " cables, tubes, connections, etc.,
" " " calibration apparatus,
labour costs in calibration of systems,
" " " installations,
" " " protection of instruments, cables, etc., during construction,
" " " reading instruments,
" " " analyses and interpretation of results.

It should be emphasised to anyone undertaking field instrumentation how important it is to protect the whole installation during construction activity, even though this may seem to be expensive at the time, and to read the instruments at frequent intervals to detect any malfunctions as early as possible. H.E.C. Tasmania experience shows that constant vigilance by one or two of the instrumentation staff on thin arch dam, concrete face rockfill dam and earth dam construction can result in nearly 100% successful operation. Attention only to instrument placing and reading may lead to anything up to complete loss of the whole system. The cost of the investment makes any appreciable loss of instruments intolerable.

Also to be emphasised is the authors' note about extreme care in calibration, installation and interpretation of results. If the difficulties of obtaining true and meaningful results in laboratory tests causes concern, as discussed above, then the possibilities of error in field measurements are so much greater that as many means of cross checking as possible are necessary to prove the validity of the results. Many engineers do not recognise this difficulty sufficiently because individual propositions appear to be so simple in concept that they tend to be thought of as "fool proof". Cross-checking different instrumentation system results or with design predictions is often chastening. Nevertheless the exercise is very necessary and worthwhile. Instrumentation can show that refinements in design have little meaning in reality, and true strain situations may only approximate those calculated. The paper indicates optimism in the expected degree of validation of prediction, method of modelling, and relevance of the material properties and initial conditions measured. The authors may be begging the question: it would be unusual if the behaviour of the materials were entirely predictable, and if the instruments themselves did not produce at least some anomalous results.

Richards' paper is an esoteric paper in that it will have value only to those who have studied the previous work on the subject of measuring negative pore water pressure. In an earlier paper (1968) Richards explains that apart from the difficulty of gravimetric water content not being a "physically continuous function" and that "extreme variations in moisture content occur over even microscopic dimensions in soil", little usefulness can be attached to moisture content results. There is no fundamental relationship between water content and soil engineering parameters. Pore pressure, on the other hand, can be regarded as a continuous mathematical variable, as it is related to the energy of retention of the water in the soil and obeys hydrodynamic and thermodynamic laws. This applies also to negative pore pressure in partially saturated soils. However there are some difficulties here, because negative pore pressure, or soil suction, is not easily measured and is comprised of several components - capillary absorption, osmotic, gravitational and thermal. Some of this is explained more fully by Aitchison (1960). Total suction is the moisture stress measurable only in terms of vapour pressure. Matrix suction is the moisture stress measurable by tensiometer, pressure membrane or suction plate. In many soils the difference between matrix suction and total suction is not significant within the range of moisture stresses commonly considered, and Marshall (1959) suggests that matrix suction may be the only significant component of suction up to total suctions of about 220 atmospheres.

The present paper concerns total suction, which ranges up to 1500 lb./sq. in. in Australian conditions, and thus can only be measured in terms of vapour pressure. A wet and dry bulb thermometer is used, and from the second law of thermodynamics soil suction is obtained from the relative humidity in the pore space.

Some field measurements have been made with the thermocouple psychrometer probes. An adjustment of the calibration curve of suction vs. instrument output in microvolts has been necessary to bring suction values to levels which experience suggests to be appropriate. Further investigation is proceeding to improve probe design and calibration procedures.

Though perhaps still not widely recognised, the possible importance of negative pore pressures has been appreciated in Australia for some time. Trollope (1961) has suggested that if the development of suction is accepted few analyses of slope failures are valid because of the neglect of this factor, and that negative pore pressure may be a fortuitous compensating factor for inacequacies in assumed stress distributions in the standard analytical methods.

Concluding Remarks:

Most measurements in the field of geomechanics give values which are only approximations or representations of complex situations. The subjects of the seven papers presented in this session are diverse, but each paper is concerned with improving measurement or instrumentation techniques, whether to obtain information where little has so far been available, to increase their accuracy and reliability, or to reduce the costs. It is by such improvement in measurement, coupled with close observation of actual behaviour, that we increase our understanding of the problems.

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Paper by C.M. GERRARD, M. KURZEME, D.C. ANDREWS and R. TOPP:

The Authors in Reply:

The General Reporter's comments are very valuable in further highlighting the sources of possible error in large field instrumentation programmes. All these sources should be borne in mind when assessing the results of field observations. The scarcity of such observations presents a danger of over-valuing the worth of the available results.

Paper by O.G. INGLES and R.C. NEIL:

Discussion by A.J. BOWLING:

As part of the Commission's investigations into the properties of mass concrete for structures such as arch dams it has been necessary to determine the variation of volumetric strain with load in concrete test cylinders. A volumetric strain gauge has therefore been devised which consists of a combination of electric resistance strain gauges bonded to a concrete cylinder. This note describes the principle of the volumetric strain gauge and outlines how it has been applied in laboratory investigations.

Principle of the Volumetric Strain Gauge:

The volumetric strain $e_v$ in a cylinder can be expressed in terms of the longitudinal strain $e_l$ and circumferential strain $e_c$ as follows:

$$e_v = 2e_c + e_l \ldots\ldots\ldots\ldots\ldots (1)$$

Longitudinal and circumferential strains can conveniently be measured with electric resistance strain gauges. It is shown below that by connecting one circumferential strain gauge in series with two parallel connected longitudinal strain gauges a circuit is obtained which will indicate volumetric strain directly.

The total resistance $R_t$ of the combination of three strain gauges is given by

$$R_t = R_c + \tfrac{1}{2}R_l$$

where $R_c$ is the resistance of the circumferential strain gauge and $R_l$ is the resistance of each longitudinal strain gauge. The change in total resistance, $dR_t$, is given by

$$dR_t = dR_c + \tfrac{1}{2}dR_l \ldots\ldots\ldots\ldots\ldots (2)$$

The equation relating strain to resistance change in an electric resistance strain gauge is

$$dR/R = k \times e \ldots\ldots\ldots\ldots\ldots (3)$$

where k is the gauge factor of the strain gauge wire.

Substituting in Eq. (2) for $dR_c$ and $dR_l$,

$$\frac{dR_t}{R_t} = k \times e_c \times \frac{R_c}{R_t} + \tfrac{1}{2}k \times e_l \times \frac{R_l}{R_t}$$

If $R_c = R_l = R_o$ then $R_t = 1\tfrac{1}{2}R_o$

and $\dfrac{dR_t}{R_t} = \dfrac{1}{3}k \quad (2e_c + e_l)$

Substituting from Eq. (1) for $e_v$

$$\frac{dR_t}{R_t} = \frac{1}{3}k \times e_v$$

The combination of two longitudinal strain gauges and one circumferential strain gauge thus forms a volumetric strain gauge with a gauge factor of $\frac{1}{3}k$.

In the present investigation two volumetric strain gauges have been formed from four longitudinal strain gauges, spaced 90° apart around the cylinder, and two circumferential strain gauges, all six gauges being located near the centre of the cylinder to avoid end effects. The two volumetric strain gauges form the two opposite active arms of a Wheatstone bridge circuit, the other two arms of the bridge being formed by temperature-compensating dummy strain gauges. This circuit is twice as sensitive to strain as a single volumetric strain gauge and the strain indications of the six individual strain gauges are automatically averaged. The out-of-balance voltage, Vd, across the bridge circuit, due to equal volumetric strains $e_v$ affecting the two active arms, is given by,

$$Vd = \frac{1}{6} V k e_v$$

where V is the bridge voltage.

Application of the Volumetric Strain Gauge:

In order to have longitudinal and circumferential strain gauges with a length sufficient to average the strains across the concrete aggregate and matrix, the strain gauges are made in the laboratory by gluing resistance wire directly to the concrete. About 40 in. of rayon-covered copper-nickel resistance wire is used for each gauge giving a gauge resistance of about 80 ohms. For the longitudinal gauges a grid is formed with a length of about 5 in. and a width of about $\frac{1}{2}$ in. For the circumferential gauges the resistance wire is wound about twice around the cylinder.

Before the gauges are attached the cylinder is allowed to become surface dry. A thin coating of epoxy glue is then applied to the central section of the cylinder to fill in any small surface holes and to form a base for attaching the resistance wire. When the epoxy glue has set the two circumferential gauges are formed by gluing the resistance wire to the cylinder with a nitrocellulose glue. These two gauges are usually about 2 in. apart. The four longitudinal gauges are then formed, again using nitrocellulose glue, a jig being used to form the wire grid before it is applied to the cylinder.

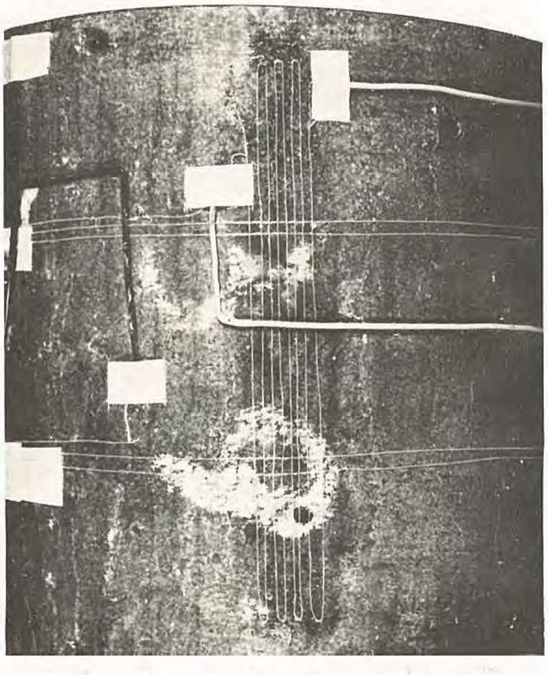

Fig. D1. - Electric Resistance Strain Gauges attached to a Concrete Cylinder.

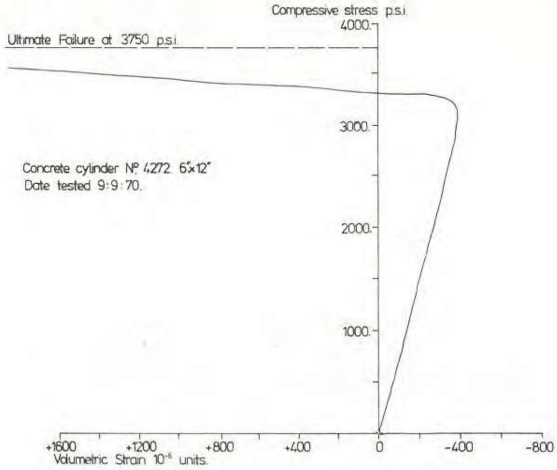


Fig. D2. - Typical X-Y Recorder Plot of Volumetric Strain against Compressive Stress.

When the glue has hardened the resistance wires are trimmed so that all six strain gauges have the same resistance. The electrical connections are then made after which the cylinder is ready for testing (Fig. D1). For testing purposes the two dummy arms of the Wheatstone bridge circuit are formed by a similarly instrumented cylinder which remains unloaded during the tests. The bridge circuit is activated by a D.C. voltage of about 4V and the out-of-balance voltage, which is proportional to the volumetric strain in the test cylinder, is fed into an X-Y recorder. A typical plot of volumetric strain against load obtained from the X-Y recorder is shown in Fig. D2.

The Authors in Reply:

To the General Reporter:

The strain measurements shown have been taken by a Schaevitz model 100HR transformer having a linear range of +0.100 to -0.100 in. with an output sensitivity of 4.3 mV per 0.001 in. displacement per volt input at 2,000 Hz. All linear strains are measured between the platens, deformations in the total system other than in the specimen being automatically deducted using stored calibrations. The radial strain measurement by frictionless band is essentially an interim technique, in which one end of the band is anchored and the displacement of the other end measured by a linear displacement transducer. Although this can be made to give an accurate mean radial (circumferential) strain for the point of application of the band, multiple bands (or a continuous frictionless wire device) are required to ensure that variations in the whole vertical profile are measured. If this is done, the calculated volumetric changes can be compared with those taken by a direct volumetric strain measurement device such as a liquid displacement gauge operated at constant pressure and temperature. As observed by the Reporter, frictionless platens are highly desirable; but even if this condition is not achieved, any non-uniform deformations can be recorded by the present techniques.

To Mr. A.J. Bowling:

The technique of volumetric strain measurement described is ingenious and simple, and we are obliged to Mr. Bowling for its description. Where very low modulus materials are involved, the two axial gauges used to average vertical strain could introduce some error; this would be negligible for higher modulus materials such as rocks, concrete and the like.

Paper by W.M. KIRKPATRICK and J.S. YOUNGER:

The Authors in Reply:

The main point raised by the General Reporter on the subject matter of our paper refers to the question of whether all strength tests should be carried out using lubricated platens. Reference was also made in the spoken report to works in which statements have been made to the effect that measured stress-strain behaviour was independent of the end-restraint condition, whether rough or lubricated, imposed at the platens in the triaxial compression test.

Our interest in this problem stems from a desire to be able to make reliable measurements of stress - strain response and we have expended considerable effort in examining conditions within strength test samples of soil. As a result of this we can only state that any experience such as that described by the statement above is completely at variance with ours.

Regarding the matter of whether lubricated platens should be used as a standard in strength tests, there are certain well established facts with reference to the triaxial compression test which can allow judgements to be made:

(1) In sands, nominal peak strengths appear to be similar whether rough or lubricated platens are used.

(2) Also in sands the stress-strain response is different depending on the end condition. Only under lubricated end conditions does a homogeneous state of strain develop.

It can be said therefore that if only peak strengths are required it does not appear to matter but if reliable stress-strain data is needed lubricated platens are essential. Since end lubrication is easily provided there does not appear to be any reason to use another.

(3) Evidence of the benefits of using lubricated platens in the testing of clays is also available. In undrained tests it has been found that non-homogeneous pore pressure measurements - reflecting non-homogeneous strain conditions which are present with rough platens are largely removed by the use of lubricated platens.

(4) We have no personal experience with regard to the influence of end conditions in the testing of rocks or other brittle materials. The criterion for desirable behaviour is however the same - that is a homogeneous state of stress and strain should develop in the samples. We would suggest, on the basis of the behaviour of soils, that if effective means of end lubrication can be found such should be used in the testing of rocks and similar materials. There appears to be a need however for research into the behaviour of test samples of these materials under different end conditions.

(5) There is less information regarding the distribution of stress within samples. Research at the University of West Virginia initiated by the senior author helps however to throw light on this matter in reference to soils. In this work reported by Seals, Newman and Kirkpatrick the distribution of normal (axial) pressure at the rigid platens of $9\frac{1}{2}$ min. dia. compression samples of Ottowa sand were measured using diaphragm gauges placed at various locations in the end platens. Tests were performed under lubricated and non-lubricated end conditions on loose and dense sands. The results were collected together to typify the conditions at three stages of the tests termed the "elastic" range at axial strains less than about 0.6%, the transition stage: $\Sigma ax$ = 1.25% to 3.75% and the ultimate range: $\Sigma ax$ = 10%.

Fig. D3 shows boundary axial stress data for tests using non-lubricated platens on dense sand in the "elastic" range. In this the ratio of the measured stress at the gauge to the average axial stress (applied boundary load divided by the overall sample area) is plotted against the ratio of the radial position r of the gauge to the maximum radius R of the sample.

Appreciable scatter of the results is evident as would be expected in this type of measurement. Regression curves for the non-lubricated dense samples at the three stages of the test are shown in Fig. D4. These curves show a non-uniform distribution with the stresses at the centre being lower than the stresses at the periphery. There is a redistribution as the test continues to higher strains with the peripheral stresses becoming larger and the stresses at the centre becoming smaller.

This non-uniform stress distribution represents a non-homogeneous state of stress within the sample

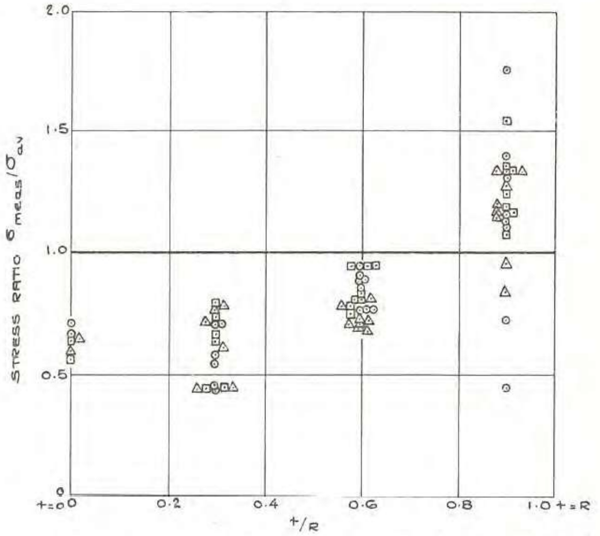


Fig. D3. - Rough Platen - Dense Samples "Elastic" Range.

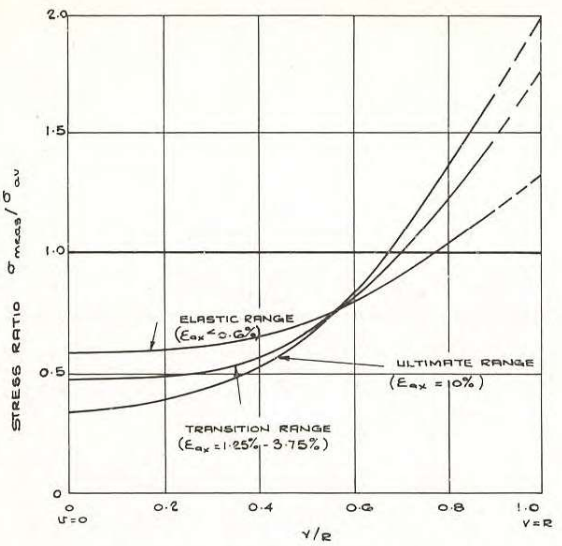


Fig. D4. - Stress Distributions over Range of Test Rough Platens - Dense Samples.

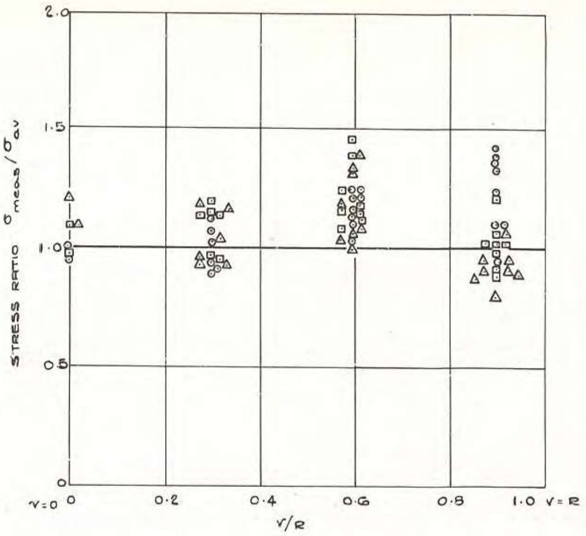


Fig. D5. - Lubricated Platen - Dense Sample - Transition Range.

and is a reflection of the non-homogeneous strain states measured in compression tests with non-lubricated platens.

Similar stress distributions were found in loose samples tested with non-lubricated platens.

Fig. D5 shows typical data for tests using lubricated platens. Again a certain scatter exists but regression analyses show uniform or near-uniform stress distributions.

Although little can be said about the distribution of stress within the sample it can be presumed that any lack of uniformity is likely to be most severe at the rigid platen boundary. If this is the case it might be possible to assume for lubricated platens that a uniform distribution of axial stress exists throughout the sample. If it can also be assumed that platen lubrication reduces end friction to negligible proportions (Rowe and Barden quote very low values for $\mu$) then the elastic solution to the stress distribution can be obtained by superposing the solution for the cylinder under purely uniform radial pressure on to the solution for the cylinder under purely uniform axial stress. This allows the inference to be drawn that the lubricated cylindrical triaxial sample supports a homogeneous state of stress with the condition that the radial and tangential stresses are equal. This refers to the elastic state but since there is no significant redistribution of axial stress nor any significant change in the strain distribution as the test advances it might be expected that the homogeneity extends throughout the entire range of the test.

It would appear from these observations of stress and strain that in sands at least platen lubrication produces a near-ideal state within the triaxial compression test and probably also in the other tests referred to in our paper.

These desirable features are absent in samples tested with non-lubricated platens. Tests involving such samples cannot be expected to yield reliable stress-strain data.

Reference:

SEALS, R.K., NEWMAN, B. and KIRKPATRICK, W.M. - Stress Distributions at the Platens of Compression Samples. In preparation.