SPECIALTY SESSION 1

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EQUIPMENT FOR THE OBSERVATION OF SETTLEMENTS AND STRESSES OF BASES

Chairman: Stanley D.Wilson (USA),
Vice-Chairmen: Georg Stefanoff (Bulgaria)
D.S.Baranov (USSR)
Participants: R.W.Cooke (England), Ch.Veder (Austria),
F.Jezequel (France), H.Seiffert (DDR),
V.Z.Kheifitz (USSR), G.Lazebnik (USSR),
N.Gruber (FRG), Vlad D.Perlea (Romania),
Y.Tcheng (France),

Chairman Mr. Stanley D.Wilson (USA)

I take great pleasure in welcoming you to the first Specialty Session of the VIII International Conference on Soil Mechanics and Foundation Engineering. The topic of this session is "Equipment for the Observation of Settlements and Stresses of Bases." The vice-chairmen are Prof. G. Stefanoff of Bulgaria and Senior Researcher D.S. Baranov of the USSR.

The main aim of this session is to acquaint those interested with the latest advances in engineering developments, related to the design and construction of instrumentation for the above purpose, the technique of measurement procedures, and the analysis of the data obtained from such measurements. Since time is limited, it is proposed to limit the discussion, insofar as is feasible, to the following two topics:

1. Up-to-date techniques and instruments for measuring settlements, horizontal movements, and tilting of foundations including foundations of soil structures.
2. Up-to-date methods of measuring stresses in foundations, or at least changes in stress resulting from the applied loads.

With respect to topic No. 2, I have informed Professor Gregory Tschebotarioff, Chairman of Specialty Session 5, that I do not propose to include the determination of the in situ stress of natural soils in this session. However there is no objection to the inclusion of discussions of this topic in this session provided the technique is also applicable to the determination of the increased stress resulting from the foundation loading.

I hope that these introductory remarks will stimulate oral discussion at this session which will be productive to further improvements in instrumentation for observing the performance of bases and foundations.

In these opening remarks I shall first give a brief review of the development of field measurement techniques. This will be followed by a review of the important points of the various papers submitted to the session. Then I shall discuss some of the needs in this area and the directions that will probably be taken by the designers of new instrumentation.

BRIEF REVIEW OF INSTRUMENTATION

Karl Terzaghi was a strong advocate of the so-called "observational methods in applied soil mechanics." This method consists of: (1) predicting in advance the performance of a structure in terms of settlement, displacement or pore water pressures; (2) making in situ observations of the parameters during construction; and (3) comparing the predicted and the observed behavior and modifying the design as required. For this method to be effective, the right observations must be made and they must be reliable. To be reliable, they do not necessarily have to be elaborate and costly. I hope that the question of reliability will not be overlooked in the discussions this afternoon of new instrumentation.

When Dr. Terzaghi first developed his observational approach, there were few available instruments that had the sensitivity, accuracy and reliability to obtain the necessary data. As a consequence he was often forced to design his own instrumentation systems and the installations were not only costly, but often failed to provide data of sufficient accuracy and reliability to permit early detection of variations between predicted and observed movements.

This situation has now changed radically as the result of new developments in instruments during the past decade. There now exist a variety of devices for measuring all three components of small movements in foundations, as well as other improved devices for measuring pore pressures and tilting. Furthermore, the availability and performance of these instruments is being made known to the practising engineer through publications and conferences such as this one.

In June 1972, the American Society of Civil Engineers sponsored a specialty conference at Purdue University on the performance of earth and earth-supported structures. The proceedings of this conference made available to the profession a large amount of data from various instruments which had been developed within the past decade. Unfortunately, a substantial period of time exists between the time a new instrument is first designed and the date records from an actual project become available. Therefore, some of the instruments described in the Purdue conference were already obsolete by the time of publication.

In May 1973, the British Geotechnical Society sponsored in London a Symposium on Field Instrumentation, at which the latest developments in instrumentation were presented. Although not intended as an international meeting, the symposium was well attended by engineers from many countries indicating the great interest in this subject.

In the USA, and I think to considerable extent in Great Britain, the present trend is for the development of instrumentation having not only the capability of greater precision, but also of being recorded automatically, or at least semi-automatically, by means of punched tape or magnetic tape. These records can then be numerically analyzed and plotted by electronic computers. It is important to appreciate, however, that the output of a computer does not constitute a critical analysis of the data. It appears to me, indeed, that the greatest deficiency at the present time is no longer in the instruments themselves, but in our capability to review and analyze the data. Perhaps it is not so much in our lack of capability as it is a lack of appreciation of the need for such critical reviews

REVIEW OF PAPERS SUBMITTED TO THIS SESSION

Twenty scientific reports from this Specialty Session have been accepted for publication in Vol. IV of the conference proceedings. These papers represent contributions from nine countries and cover a variety of types of measurements, which in general can be subdivided as follows:

Type of Measurement	No. of Papers
Horizontal displacement or vertical bending	4
Vertical settlement	8
Pore pressure	2
Total soil pressure	7
Strain	4

With respect to horizontal movements or deflections of foundations, precise surveys are a completely satisfactory procedure provided reasonable care is taken to obtain the necessary precision. When the distribution of horizontal movement with depth is required, a precision inclinometer, such as that shown in Fig. 1, normally provides sufficient accuracy which is of the order of 5mm per 30m of depth.

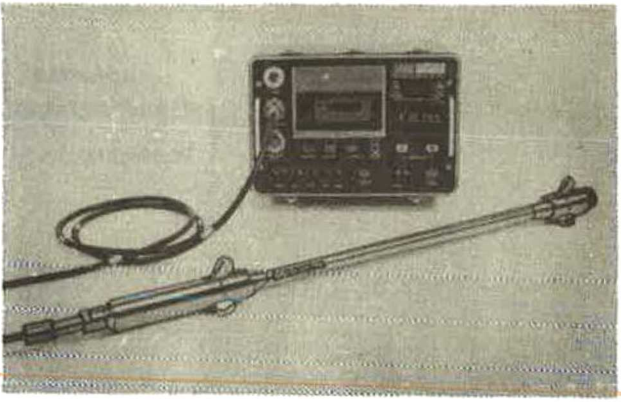


Fig. 1. Digitilt Inclinometer with Magnetic Tape Recorder

The inclinometer shown in Fig. 1 has a control box which records the inclination, depth, and other pertinent data on magnetic tape. The output of this tape may then be converted directly into digital readings or read into a computer which is programmed to provide plots of deflection with depth.

In one of the papers to be included in the proceedings, Jamiolkowski and Marchetti from Italy describe an example of the use of the Digitilt Inclinometer in connection with lateral load tests on piles, in which they evaluate the variation of soil modulus with depth.

Escario from Spain describes an optical method for measuring the deformation of concrete diaphragm walls for which an accuracy of better than 1mm in 15m of depth is claimed.

Settlement, of course, is of major importance in evaluating the performance of bases and foundations. As with horizontal movements, precise surface surveys are often the most inexpensive way of obtaining the required information. Gerrard and Kurzeme from Australia describe surveys to measure settlements of raft foundations under multi-story buildings. The distribution of settlement with depth was determined by means of anchor plates with isolated rods extending to the surface.

A number of new devices have been developed for the measurement of settlement. R. & E. Diamenti from Italy used photoclinographs obtained from bifilar suspended horizontal pendulums with great sensitivity, to measure small tilting of an important Italian monument while grouting of the foundation was being accomplished.

Peignaud from France describes an apparatus having a compressed carbon dioxide-water interface which makes it possible to measure settlement as large as 6m with an accuracy of 1cm, and this accuracy can be improved for structures. Lamasson and Carlo, also from France, describe a unique hydraulic settlement amplifier for which they claim the accuracy of measurement was of the order of 0.10mm for 2cm settlement ranging over two years. Certain compensating circuits must be used to minimize temperature effects.

Overflow weirs or inverted siphons have been used for many years to measure settlements in dams. Tominaga and Echigo from Japan present data on settlements of an ore storage yard, using such devices.

Burland describes a simple borehole extensometer developed at the Building Research Station in Great Britain. This device consists of buried circular magnets which surround a central guide tube, and which actuate a sensor containing a reed switch attached to a steel tape. Accuracy of about 0.5mm is reported.

Jewsbury of the USA describes a different type of vertical extensometer consisting of linear potentiometers connected by steel rods to anchors at various depths in a borehole. The device is particularly well adapted to measure the vertical distribution of heave during foundation excavation, as well as the subsequent settlement.

Baranov, et al of the USSR investigated the effect of the footing configuration and of the preparation of the sand base by means of 150cm diameter circular load plates and by 60x300cm rectangular load plates. Pressure cells with hydraulic transformers were used.

Kheifits, of the USSR, summarizes the results of extensive testing at the Scientific Research Center of Hydroproject Institute and on the basis of new specifications, a detector has been constructed which comprises a thin rubber sheet penetrated by channels filled with a low compressibility fluid. Hopefully, this new pressure cell will prove to be far more selective for normal stresses than rigid disc-type detectors.

Kleshev and Muller, also from USSR, describe their experiences in measuring the interaction of a building with a settling foundation, using stringed dynamometers between the footings and the building.

Mindich and Vyalov, USSR, used 33mm diameter pressure gages with an unique sensing element to investigate the stresses developed in a layer of clay on a rigid base, when subjected to loads from a rigid strip-type footing.

Although pore pressure observations are often an essential part of any foundation instrumentation program, no really new developments are reported. Perhaps this is because there are available a variety of devices, which, when properly installed, will measure pore pressures with acceptable accuracy.

Silva, from Mexico, provides details of a pneumatic type pore pressure transducer having a stainless steel diaphragm.

The determination of in situ stresses prior to new construction is not considered a valid topic for this session, however, the measurement of stresses imposed by new construction is desirable in many instances and a number of authors describe either the application of available stress cells to specific projects, or the development of new sensors.

Peignaud, from France, describes pneumatic total stress cells of 50 and 215mm diameter with a diameter deflection ratio of the order of 5000, which can be increased to 10,000 in continuous operation with slight permanent leakage of air past the valve.

An application of the well-known Gloetzl cell is given by Haws and Tabb, Great Britain, who also describe 8000kn capacity pile load cells which utilize eight individual photoelastic load columns.

The importance of measuring ground strains has been emphasized in papers by several contributions to this session, including Titov and Chromov, USSR, who describe an apparatus for measuring resilient settlements in highway subgrades. Dietrich and Salley, USA, describe horizontal dynamic strain measurements under an embankment.

Mursenko and Revenko, USSR, describe an automatically controlled testing complex for experimental investigations of foundations.

Miller and Brown, describe remote reading instrumentation used in conducting large plate bearing tests against the vertical side faces of an elongated test pit at a depth of 58 feet below ground surface. Loads of up to 1000 psi were applied to 34-inch diameter Freyssi flat jack flexible pads. Maximum deflections at the center of the pad of 0.10 inches were obtained, and all data were recorded at the ground surface.

FUTURE DEVELOPMENTS FOR INSTRUMENTATION FOR BASES

The two greatest needs for improvements in instrumentation for bases and foundations are, in the opinion of the chairman, related to (1) greater use of automatic recording, especially with regard to deep excavations, and (2) improved techniques for installing instruments to ensure that the observed behavior of the ground is not caused by the method of installation.

With respect to increased use of automatic recording equipment I would like to use several illustrations. Fig. 2 is the control console containing semi-automatic recording equipment which monitors the stability of a reservoir slope. The output from each of the sensing elements, which are in boreholes up to 500 feet in depth over an area 3000 feet in length and 2000 feet up the slope, is conducted through electrical cables to the central control panel shown in the illustration.

Fig. 2. Master Control Console for Slope Stability Instrumentation Monitoring

Here the data from 134 extensometers, 14 rock noise detectors, and 55 electrical piezometers are centrally recorded. The data can then be put directly onto a teletype and transmitted to an office several hundred miles away for analysis and review, all within a matter of minutes.

An even more completely automated system is shown in Fig. 3. Referred to as an in-place inclinometer, this device consists of multiple accelerometers mounted on series-connected rods installed in near-vertical boreholes.

Changes in inclination from the vertical are detected by a master control unit, Fig. 4, which samples as many as 20 sensors each 2 seconds. The changes can either be observed visually or recorded in any of several different ways. Furthermore, by means of a long-distance telephone and the unit shown on Fig. 5, it is possible to obtain in a few seconds a print-out tape of the changes at any desired time, even from locations several thousands of miles distant.

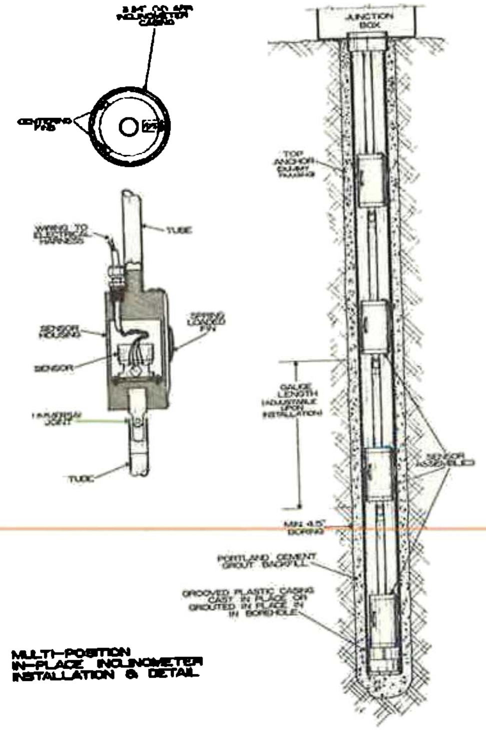


Fig. 3. In-Place Inclinometer Sensors

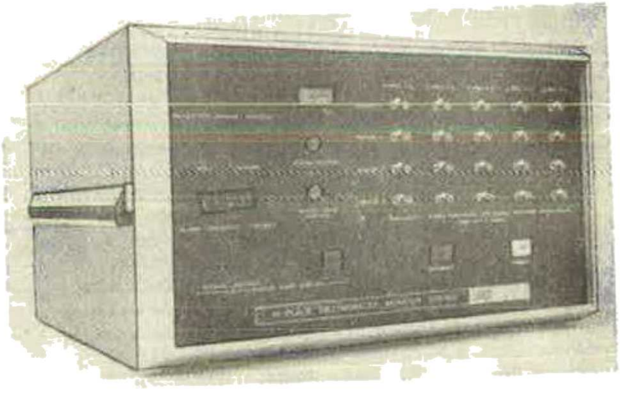


Fig. 4. In-Place Inclinometer Console

Fig. 5. Remote In-Place Inclinometer Monitor

With regard to installation techniques, many improvements are needed, particularly with regard to total pressure cells. Regardless of the capability of the cell itself, the readings may be completely erroneous unless the backfill immediately surrounding the cell has identical properties with that of the rest of the embankment.

SUMMARY AND CONCLUSIONS

The papers submitted to this specialty session include a variety of new instrumentation and improvements to well established types of instruments. It is significant to note the increased emphasis being placed on instrumentation to observe the performance of bases and foundations.

These papers demonstrate that there are in existence today, techniques and instruments for measuring settlement, horizontal movements, tilting, pore pressures and strains in foundations. Further improvements will occur particularly with regard to reliability, ease of installation, and increased use of automatic recordings.

With regard to measuring stresses in foundations, the state-of-the-art is not so clear. Progress is being made, but there does not yet exist a simple way of installing such devices so as to obtain uniformly reliable data.

Chairman Mr. Stanley D. Wilson

Ladies and Gentlemen I wish now to invite the delegates to take part in the discussion. Mr. Baranov, please, deliver your contribution

D.S. Baranov, Vice-Chairman, (USSR)

Mister Chairman, Ladies and Gentlemen,

The problem of strain measurements in foundations and footings can now be considered as principally solved and further improvements in reliability of the results are mostly a matter of an appropriate design of the test instrumentation.

Stress measurement is a much more complicated problem closely connected with a methodology of estimation of the interaction between soil and measuring device which can be assumed as an inclusion into a soil.

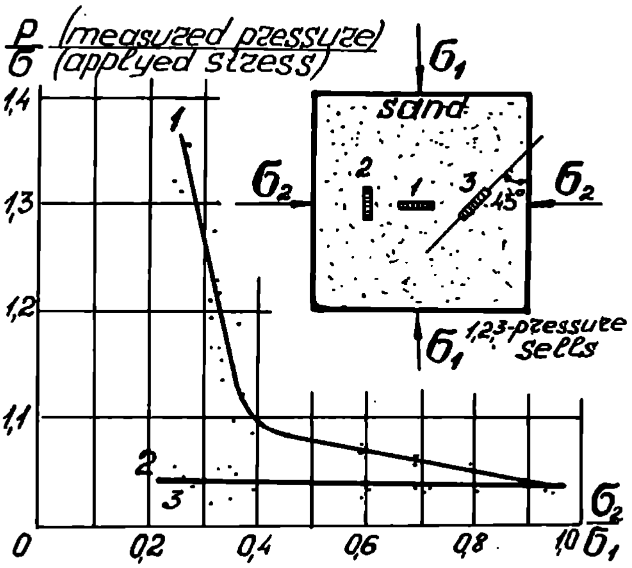
Some investigations carried out in the USSR in recent years clearly showed that neglecting this interaction can result in noticeable errors of measurements /1/.

Here I should like to dwell on some principal methodology aspects of the problem.

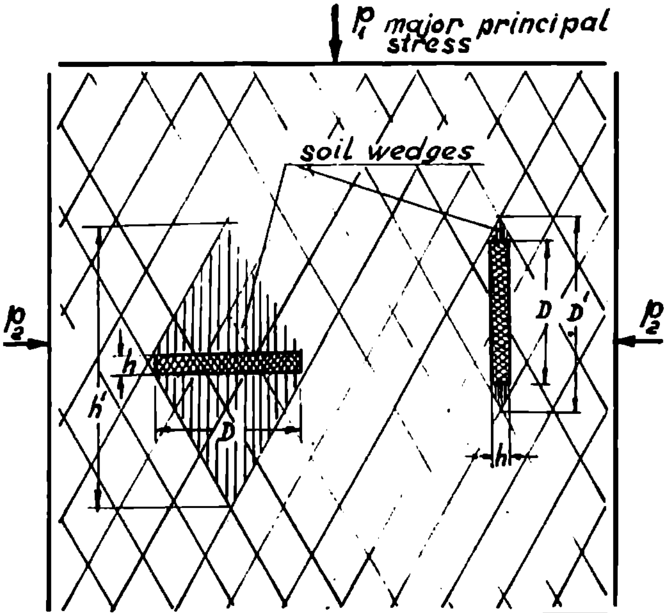
It has been proved experimentally that maximum reliability of the stress measurements can be reached only with comparatively rigid gauges. This principle is of particular importance when measuring varying and dynamic stresses. But in the case of a rigid gauge involved the stress concentration normally takes place in a surrounding soil, its rate being in a direct proportion to the ratio of height h to diameter D of the gauge. It has been found that in soils with high angle of internal friction the stress concentration is asimmetric. This asimmetry as a result of negative friction acting under the conditions of soil consolidation can reach 25-30 per cent of the measured stress value. The maximum asimmetry takes place when measuring stress in the direction of maximum soil deformation while in a normal to direction the asimetry is minimum.

The value of stress concentration in a surrounding soil depends also on a general stress state of the soil or, to be more correct, on a principal stress ratio σ_2/σ_1. In a figure 1 are shown the results of the tests of pressure sells in sand at various ratios σ_2/σ_1. With approaching by the soil stresses a critical level, the stress concentration along the σ_1 direction steeply grows as a result of formation of undeformable rigid areas in surrounding sand which increase the equivalent height of the gauge (Fig.2). This growth of the stress concentration has not been observed for the gauges measuring σ_2 and $\sigma_{45°}$. The described above data show that results of gauge calibrating tests under uniaxial compression can not be considered as sufficient. The test arrangement should provide more realistic loading conditions.

Effect of σ_2/σ_1 ratio on stress concentration for sands



The last aspect, I should like to draw your attention to, is a substantional influence of soil non-homogenity on measurement



Effect of sizes increasing of soil pressure sell while approaching to the ultimate state

results. Non-homogenity can be produced either by some irregularities in filling (up to 25-30%), or in the process of the gauge installation. For example, even a slight grinding of the gauge down to the sand resulted in noticeable (up to 20%) increase in variation of the readings. Influence of the soil non-homogenity should be particularly taken into account in field measurements where there is often no opportunities to repeat the tests.

The special technique for stress measurements in soils has been developed in the USSR in recent years which allows to obtain a high level of reliability by means of the aplication of the method substantional decreasing of stress concentration in a surrounding soil. This allows to determine experimentally all of the components of the stress tensor.

A strain gauge pressure sell developed at the Central Research Institute for Building Structures (GOSSTROY THE USSR), has pasted a complex of metrological tests /2/ and is success fully used for laboratory and field tests including dynamic and long term tests.

There is an important methodological problem which need a profound study. That is the development of a method for pressure sell insertion into natural structure soil. As the investigations have shown the pressure sell insertion into a bore-hole followed by its tamping causes considerable disturbance of the natural state of soil stress condition round the sell and which in most cases does not return its former state. Pushing the pressure sell into the soil also distorts on soil stress condition and the measured date may be increased up to 500%.

It is neccessary to find the avoid the distortions of natural soil stress condition as much as possible.

Chairman Mr. Stanley D. Wilson

Thank you Mr. Baranov for your interesting contribution. The next will be Mr. Cooke (England)

R.W. Cooke (England)

EQUIPMENT FOR THE OBSERVATION OF SETTLEMENTS AND STRESSES OF BASES

Inclinometers measuring deviation from the vertical are now used to detect the deformations of sheet pile walls, the lack of straightness of long, small-displacement piles and earth movements occurring close to deep excavations. As far as I am aware there is no published data on inclinometers for measuring small angles to the horizontal such measurements being capable of integrating with respect to length to give values of foundation settlement or vertical ground movement. At the Building Research Station in England we have developed a range of horizontal inclinometers for this purpose, Cooke, R W and Price, G (1973), and observed vertical ground movements around and beneath experimental piled foundations. Two of the instruments were used to provide the data described in our paper to this Conference, 'Strains and Displacements around Friction Piles'.

The inclinometers, which are based on electrolytic liquid levels, are used in horizontal boreholes lined with plastic tube, 50 mm in diameter, drilled from a trench outside the soil zone affected by the loaded foundation. The resistance between three electrodes partially immersed in the level electrolyte changes as the tube containing the electrolyte is tilted in a vertical plane. By using precise resistance measuring bridges, sensitivities better than 5 x 10^{-6} radians (or 1 second of arc) can be detected. A section through the head of one model of the inclinometer incorporating a sliding wedge for initially zeroing the instrument is shown in Fig 1. Adjustments of the wedge are obtained by rotating a drive shaft which passes through a universal coupling to terminate in a fine screw-thread at the thicker end of the wedge.

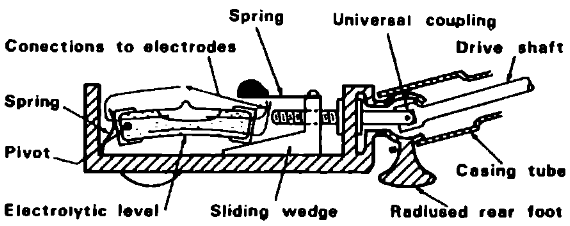


FIG. I

The distance from the observation trench over which this instrument can be operated is limited by the length of the drive shaft that can be conveniently formed and by the width of the trench used to install the inclinometer. This difficulty has been overcome in two subsequent developments. In the first of these the drive shaft is eliminated and one end of the level tube rests on a cam driven by a remotely-controlled, low-speed motor. The instrument can therefore be pushed to any position in the borehole by a flexible rod or pulled by a cord from a subsidiary trench at the opposite side of the foundation under study. This model of the inclinometer, Fig 2, was used to obtain the curve of vertical soil displacement beneath the pile point shown in Figs 5a and 5b of paper 3/9 in vol 2.1

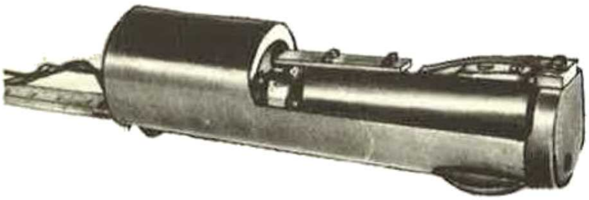


Fig 2

The most recent work has been carried out with trains of simpler inclinometers (Fig 3) incorporating electrolytic levels having a much larger operating range. Because of the extended range, no method of zero balancing is required as long as the borehole and lining tube can be installed to within a few degrees of the horizontal.

Fig 3

Once installed the trains of inclinometers remain unmoved for the duration of the experiment and errors associated with moving and replacing instruments in the same position are avoided. Individual trains of up to 13 inclinometers are being used. They are connected by universal couplings and are supported on feet radiused about the coupling centre. Accurate measurements of the soil displacement profiles around groups of two and three instrumented piles are being obtained and some data for a two pile group will be presented in Main Session 3 of the Conference.

REFERENCE

Cooke, R W and Price, G (1973). Horizontal inclinometers for the measurement of vertical displacement in the soil around experimental foundations. Proc Symp Field Instrumentation in Geotechnical Engineering. Butterworths, London, pp 112-125.

Chairman Mr.Stanley D.Wilson

Thank you very much, Mr. Cooke,

then I pass the work to Mr.Veder

from Austria

Christian VEDER (Austria)

The soil under a freight yard in Western Austria consists of layers of peat and silt, with some intermediate layers of fine sand. Only at depth from 6 to 11 meters is gravel found. It appeared highly important and necessary to measure not only the total amount of the settlement due to the preconsolidation load as a whole but also its distribution respective to the depth. Preliminary experiments had proved the insufficiency of conventional telescopic depth gauges. Therefore, a new method of measuring the settlement at different depths was applied: by means of drilling equipment, measuring plates (Fig.1) were drilled into the soil at different depths. A jerky backward movement of the drilling rod released the bayonet fitting (or scew cap), and the plate remained in the soil at the desired depth. To keep the drilling hole free for the maesuring sound, a flexible plastic pipe was left in the ground,going trough the middle of the plates without being attached to them. Thus the plates (Fig.2) could follow every vertical movement of the different layers of soil. The measuring sonde fixed to a calibrated cable, consists of a plastic covered coil winding which produces an electrostatic field. If this electrostatic field is changed by one of the iron plates, an indicator at the other end of the cable shows a deflexion of pointer. Thus the depth of the different plates can be exactly determined with an accuracy of plus/minus 1 mm.

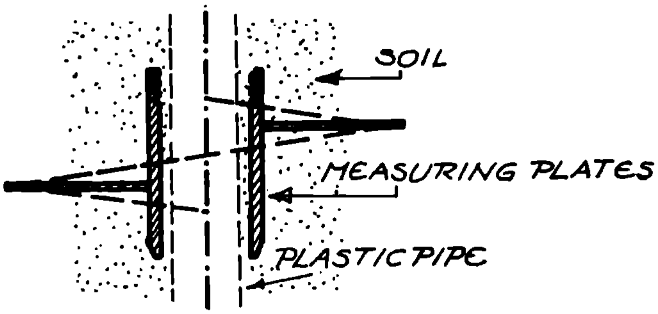


FIG.1 MEASURING PLATE

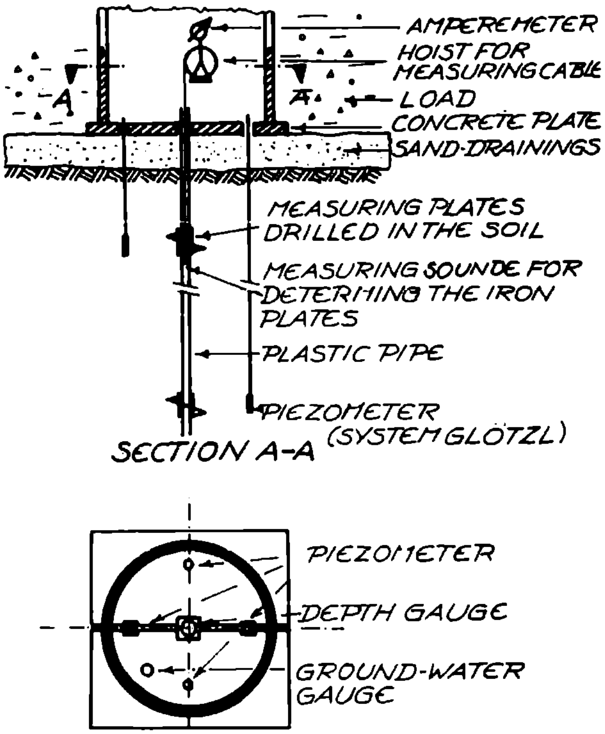


Fig. 2. Position and measuringsystem of the depth gauges

Chairman Mr. Stanley D. Wilson

Thank you very much, Mr. Veder for your contribution .

The next will be Mr. Jezequel, France

Jean François JEZEQUEL (France)

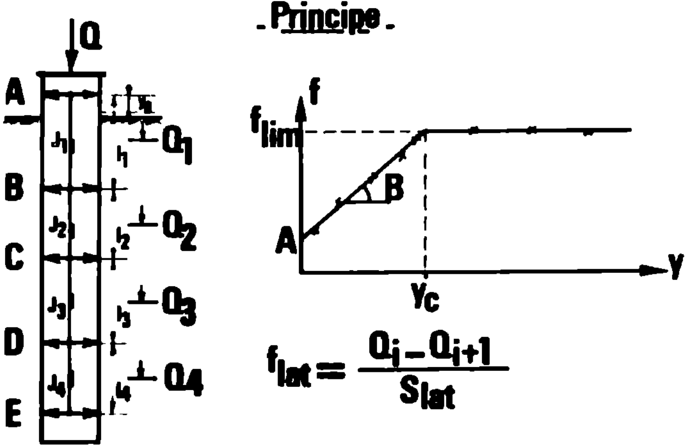
La mesure des relations contraintes-deformations dans les futs des pieux charges statiquement necessite la mise en place de moyens de mesure delicats et onereux.

On presente ici un appareillage nouveau, experimente en France depuis deux ans (JEZEQUEL et autres 1962) et dont l'interet essentiel est d'etre recuperable apres mise en oeuvre dans n'importe quel type de pieu.

1. Principe

Un ruban metallique portant des jauges de deformations collees est tendu entre divers points d'un pieu a l'aide de dispositifs speciaux appeles "bloqueurs" (figure 1). On determine ainsi des tronçons de pieux de longueur variable AB, BC ... .

Fig.1 - Extensomètre amovible - Principe -



$$f_{lat} = \frac{Q_i - Q_{i+1}}{S_{lat}}$$

$$y_i = y_0 - [\Delta l_1 + \dots + \Delta l_i]$$

Sous l'action de la charge (Q) le pieu se comprime. La compression $/\Delta \ell/$ de chaque tronçon elementaire de longueur $/\ell/$ est donnee directement par la jauge collee sur le ruban.

2. Realisation pratique

La mise en oeuvre du dispositif necessite la mise en place prealable d'un ou de plusieurs tubes logements de 5 cm de diametre interieur. Les tubes sont: soit noyes dans le beton, soit soudes directement sur les pieux metalliques. Cette operation ne presente pas de difficulte pour les pieux H. Elle est plus delicate pour les pieux tubes non visitables de grande longueur. Dans ce dernier cas on peut d'ailleurs utiliser une autre methode decrite dans la reference.

Un train de rubans et de bloqueurs est descendu par simple gravite dans le tube logement.

Le bloqueur, situe juste au-dessus de la pointe du pieu, est alors dilate seul grace a un circuit pneumatique independant.

L'ensemble du train de rubans est alors mis en traction depuis la surface, traction controlee par les jauges et dont le niveau est fonction du type d'essai de pieu (chargement ou arrachement).

Les autres bloqueurs sont ensuite dilates simultanement grace a un deuxieme circuit hydraulique et l'ensemble est alors pret a fonctionner.

3. Quelques points technologiques

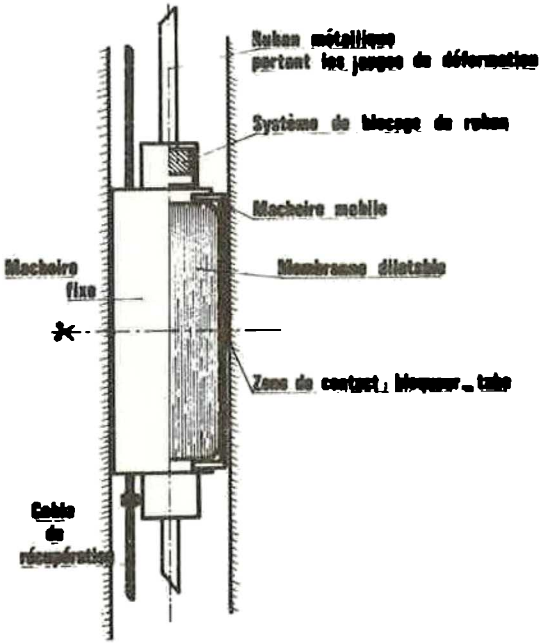
Le succes de la methode est lie au respect d'un certain nombre de precautions technologiques:

- le ruban doit etre de caracteristiques geometriques uniformes, faute de quoi la mesure a la jauge de deformation ne sera pas representative de la deformation d'ensemble et un etalonnage sera necessaire.

  Le module du ruban et sa section doivent etre aussi faibles que possible afin que sa variation de longueur n'induise que de faibles variations de forces sur les bloqueurs.

  Par contre sa limite elastique doit etre elevee afin de pouvoir extraire l'ensemble sans difficulte sans avoir recours au cable de sauvetage (fig.2).

Fig.2 Bloqueur: Schéma de principe. Prototype n°2



- La conception des bloqueurs est particulierement importante puisqu'ils assurent la liaison rubans-pieu. La surface de contact bloqueurs-pieu doit etre faible afin d'eviter les compressions parasites de contact.

Grace a ce nouveau procede il a ete possible de diviser par deux le prix de ces mesures car la mise en oeuvre est rapide et le materiel recuperable.

La mesure n'est perturbee par aucun phenomene de fluage ou de relaxation et, dans les betons, elle conduit a des resultats plus fiables que ceux obtenus par les extensometres classiques car la base de mesure est bien plus importante.

La mise en place de l'appareillage ne necessite pas de preparation particuliere de la tete de pieu - a l'exception d'une echancrure pour le passage des fils-. Elle peut se faire avec un espacement reduit d'un metre environ sous le massif de reaction.

Mais le procede, tel qu'il a ete decrit, n'est pas adapte aux essais a longs termes (etude du frottement negatif par exemple) en raison de la necessite d'alimentation des circuits pneumatiques.

REFERENCE:

J.Jezequel, E.Lemee, J.N.Guegan et P.Liberge. "Appareillage amovible pour la mesure des relations contraintes-deplacements dans les pieux". Bulletin de Liaison des Laboratoires des Ponts et Chaussees No.57-Janvier-Fevrier 1972

Chairman Mr.Stanley D.Wilson
Thank you very much Mr.Jezequel for your interesting contribution.
Now, Mr.Seiffert, please,

Seiffert, Horst Dr.ing (DDR)

At present the construction of technical devices is being more and more improved; therefore the measuring equipment for soil stresses has to meet two essential conditions, offering great difficulties:

During sustained loading the measuring instruments have to indicate constant calibration characteristics over a very long period, for instance over one year or even several years and, moreover, their installation must not disturb the stress conditions in soil.

Some years ago the team Dr.Martin, Dr.Seiffert, Assmann of the "Forschungsanstalt für Schiffahrt, Wasser- und Grundbau" developed a pneumatic measuring equipment, based upon the principle of compensation; meanwhile we have collected sufficient experiences with it /1,2,3/. This equipment measures directly the mechanical stresses in any medium. The external stresses are compensated by pneumatic pressure in the following method: compressed air, coming from a pressure tank, passes through a pressure reducing valve, flows into a main line, then through a nozzle into a supply-line to a double-membrane valve-gauge containing a pressurerelief valve (Fig.1)

The relief valve is closed by the external stress which is to be measured; the pressures set up in the lines are graded such that a constant raise of pressure takes place in the valve-gauge until the pressure-relief valve opens. Then, when the valve is open, a stationary flow forms, and the external stress is compensated by the pressure in the valve-gauge and indicated on a pressure-gauge as a dynamic pressure, analogous to the hydraulic compensation. The air flowing off the gauge is conducted through a discharge line into a glass filled with water, so that an operation control is possible by observing the airbubbles. The equipment can be installed with any member of gauges.

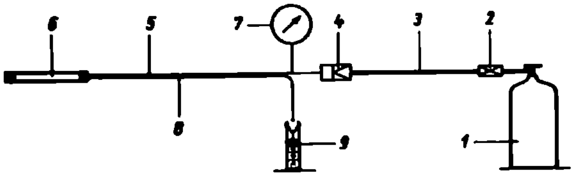


Fig. 1 System of the pneumatic measuring device

Analogous to the well-known simple valve-gauge operated hydraulically, we developed a double-membrane valve-gauge (DM-VG) (Fig.2 and 3). Due to its relief pressure-valve, which is elastic and acts in a laminar way, it shows an operation security of valve action contrary to the gauges operated hydraulically. Therefore it is suitable for the use of air as a pressure compensating medium. By containing two membranes the gauge can move in both directions. Stress conditions in soil are less disturbed and detected more correctly.

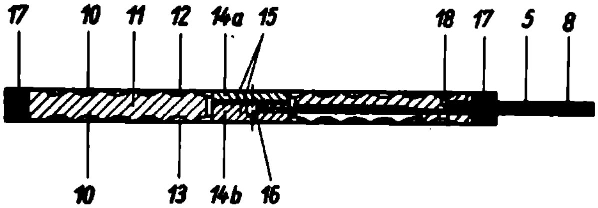


Fig.2. Section of a double-membrane valve-gauge (DM-VG)

Legenda (Fig.1 and 2)

1 pressure tank
2 pressure reducing valve
3 main line
4 nozzle
5 feeding line
6 double-membrane valve-gauge
7 pressure gauge
8 discharge line
9 water-filled vessel
10 grooves
11 supporting body
12 membrane
13 membrane
14a lamina
14b lamina
15 relief valve
16 hole
17 overhanging edge
18 hole

For several years pressure cells type DM-VG (40 and 70 mm Ø and 2,5 mm high) with flexible synthetic lines ($Ø_a$=3mm, $Ø_i$=1.5mm) were installed for model measurements. They were exposed to sustained load for a period of more than three years. After this time they showed calibration characteristics superior to all known electrical measuring instruments.

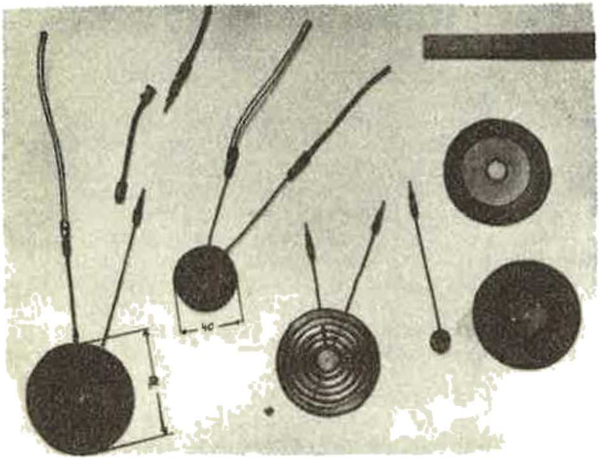


Fig.3. A double-membrane valve-gauge with its constructional parts

For estimating the quality of the measured values, in all tests the applied load was compared with the measured soil stresses, using different load increments and removals. Differences of 0.0 to 8.0% were found.

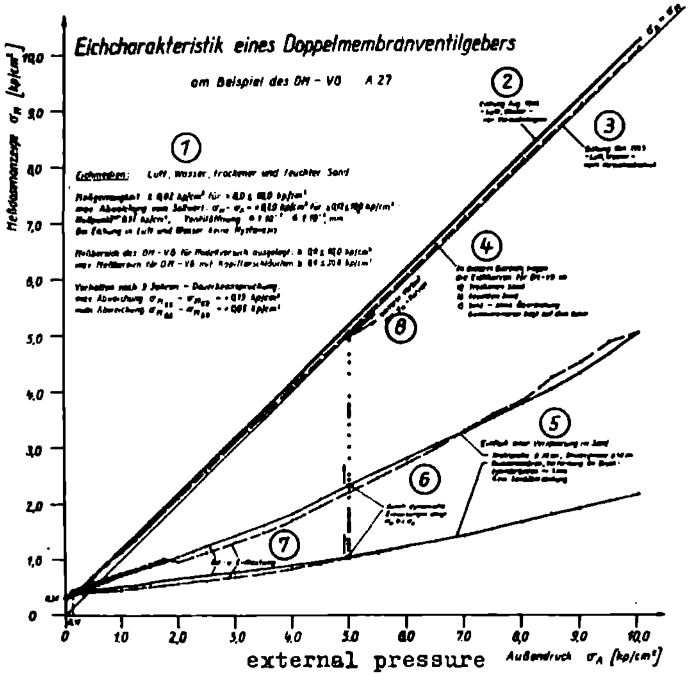


Fig.4. The calibration characteristics of a double-membrane valve-gauge

Legenda:

1) Calibration media: air, water, dry and wet sand measuring accuracy ±0.02 kp/cm^2 for $> 0.0 \leq 10.0$ kp/cm^2 maximal deviation from nominal value: $\sigma_M - \sigma_A = +0.20$ kp/cm^2 for $0.12 \leq 100$ kp/cm^2
   zero point: 0.31 kp/cm^2, valve operating $\geq 1x10^{-3} \leq 1x10^{-2}$mm
   No hysteresis on calibration in air and water
   Measuring range of DM-VG for model tests: $\geq 0.0 \leq 10.0$ kp/cm^2
   maximal measuring range for DM-VG with capillary hoses: $\geq 0.0 \leq 30.0$ kp/cm^2
   Behaviour after sustained loading over 3 years:
   maximal deviation $\sigma_{M66} - \sigma_{M69} = +0.15$ kp/cm^2
   average deviation $\sigma_{M66} - \sigma_{M69} = +0.15$ kp/cm^2
2) Calibration August, 1966
   - air, water -
   before test begin
3) Calibration October, 1969
   - air, water -
   after test finish
4) In this range the calibration curves for DM-VG lie in
   a) dry sand
   b) wet sand
   c) uncovered sand
   rubber membrane lies on on the gauge
5) Influence of a wedging in sand steel plate Ø30 cm, pressure cylinder Ø 40cm rubber membrane, deformation in bottom of pressure cylinder ~ 1 mm, 4 cm sand cover
6) σ_M to σ_A rises due to dynamic actions
7) loading and removal of load
8) further run of σ_M- curves

Some essential specialities of calibration characteristics are represented on fig.4, taking a valve gauge as an example. There also is to be seen its behaviour after a sustained load of three years. Besides we refer to two interesting results of the calibration tests, represented on fig.4. In a pressure cylinder we measured wedging effects in the soil, which could be removed by external dynamic actions, as for instance knocking. They were caused by small irregular degrees of compactness. The example shows, how much the stress in soil depends upon slight differences of compactness. They must be supposed under every footing. It may be mentioned that the constructional measuring accuracy of devices, amounting to ±0.02 kpcm^{-2} (fig.4), depends above all upon the accuracy of pressure gauge measurement.

The measuring equipment, designed upon the principle of compensation with valve gauges, is, with constructional modifications, also adaptable for measuring the compression stresses in soil, concrete, gas, water, and the pore water pressure.

The mentioned team developed a shearing stress pressure cell, which works as a valve-gauge on the above described measuring principle of pneumatic compensation. The shearing-stress, acting horizontally on the surface of a flat pressure-cell, is compensated by pneumatic pressure through an elastic meander-shaped hose, which is arranged horizontally in the pressure-cell, and by a valve. The measured value is indicated on a pressure gauge in the same way as on the DM-VG. The valve opening, the extension of the hose, and the movement of the pressure-cell surface effect a compensation of the shear stress, which is to be measured. Movements are in the range of 1x10^{-3} to 1x10^{-2}mm. The accuracy of measurements equals that of the DM-VG.

The tests performed in the "Forschungsanstalt für Schifffahrt, Wasser- und Grundbau" have shown that the pneumatic measuring equipment meets the measuring requirements, which are necessary to perform stress measurements in soil.

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Chairman Mr. Stanley D. Wilson

Thank you Mr. Seiffert for your discussion.

Now please Mr. Heifitz, USSR

V.Z. Kheifits (USSR)

The output signal of a stress detector always reflects the averaged, in a certain way, value of stresses acting on its sensing element area and being distorted by the detector itself.

So the output signal, being a random value, can be characterized by the value of its mathematical expectation and coefficient of variation. The coefficient of variation can be calculated beforehand if we know dispersion and autocorrelation function of the deformation properties of a soil mass or its density, the dimensions of the detector and the influence surface area of its sensing element.

The value of distortion contributing to the main part of the mathematical expectation shift and resulting from the stress concentration on the detector depends upon the extent of difference between the deformation characteristics of the detector and the soil displaced by it, upon the soil stressed state and the loading condition including the procedure of the detector installation.

The distortion value can be predicted only on the basis of the results of the detector testing under the known beforehand stressed states, at different loading conditions. The tests should specify one of the most important metrological detector characteristics- its sensitivity to variegated physical conditions of its interaction with soil, to the principal stresses ratio, strains, loading conditions, stress gradients (which are sometimes called "effect of orientation"), etc.

Statistical processing of results of many years of studies carried out by the Scientific Research Centre of "Hydroproject" revealed that the coefficient of variation of the measured in the field values of the average stresses on the detector, about 150 mm in dia, equals 0.3-0.5. This value can be decreased only by increase of the detector size. For reliable forecasting of the required increase of dimensions, it is necessary to accumulate knowledge about the dependance of the nonuniformity of the soil deformation properties on the soil type and procedure of the soil placement.

However the determination of the sensitivity of large detectors to different conditions of their interaction with soil is a time consuming procedure requiring bulky installations. But this limitation is easy to be avoided by testing a detector model, because the stress concentration on the detector does not depend on its size. The test devices used can be not very large, with the soil specimen volume from 0.03 to 0.05 m^3.

Model tests should comply with two requirements:

- creation of boundary conditions providing high soil specimen uniformity in the testing apparatus cell;
- ensuring geometric similitude and complete conformity of the deformation properties of the detector model and prototype.

Experience in development of apparatuses providing uniform stressed-strain state (with central symmetry in any point) in a soil specimen has demonstrated, that only in an apparatus with symmetric boundary conditions, which exclude strain and stress gradients formation at the specimen boundaries, it is possible to achieve reasonable uniformity, one of the criteria of which being the absence of the "effect of orientation". For this, the cell walls contacting with the soil should deform together with the specimen and the condition of the walls interaction with soil should not depend on the specimen strain.

Such apparatuses were used for testing several soil stress detectors of various types which have different ratio of principal dimensions and different deformation properties. Among the detectors tested, thin low-compressibility non-rigid rubber detectors (their thickness to diameter ratio is less than 0.03) produces the least distortions in the widest range of stressed state and loading conditions. Their errors did not exceed 15%, irrespective of the type of the stressed state or loading conditions.

Chairman Mr.Stanley D.Wilson

Thank you Mr.Kheifitz. Now, please, Mr.Lasebnik, USSR

G.Lasebnik (USSR)

At present time there are various opinions about values and character of errors of earth pressure cells for measuring compressive stresses in mass of soils and other loose substances.

As we are engaged in calculating and designing of initial measuring devices,we checked the error of flat cells.The check was carried out for the stress-strain state of soil, ordinarily occurring under footings. We investigated the "simple" trajectory of loading, that is the constant value of ratio $\sigma_r : \sigma_z$ at the following states of soil (medium-granule sand): I.Uniaxial compression along axis Z without the possibility of lateral expansion; 2. Uniaxial compression along the same axis with a degree (up to $\varepsilon \leq 2.10^{-2}$ of expansion of the specimen. The check was carried out in a laboratory set up 47cm in diameter at stresses σ_z from I to 4kg/cm^2 and in a large testing tank 5m wide and 5m deep under a round stiff footing at σ_z from I to 5 kg/cm^2. Ratio $\sigma_r : \sigma_z$ in the laboratory set up without the possibility of lateral expansion of the soil is within the limits of 0.57 to 0.53; the ratio with the possibility of expansion constitutes 0,44 to 0,38. The values of radial stress σ_r were measured with nine earth pressure cells, type MCM-2.

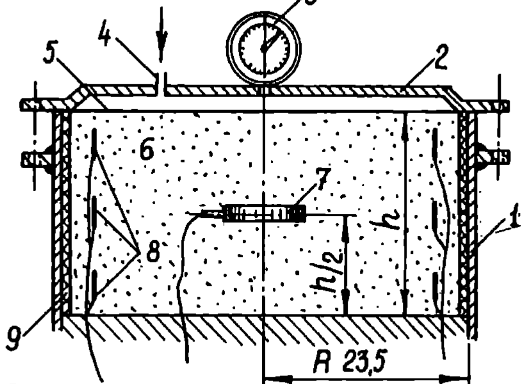
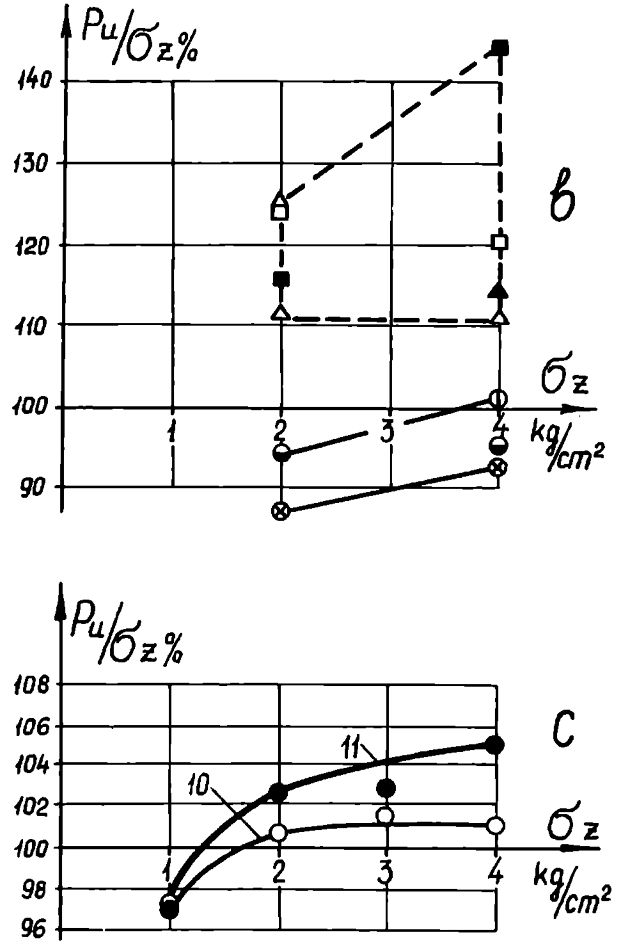


Fig.1a. Diagram of laboratory set up and results of the errors of earth pressure cells in the laboratory

a -schematic section of setup with soil and equipment installed

Cells of various design, the error of which was determinated,wre placed one by one in the centre of the soil specimen (Fig.i,a). The sensibility of cells in the tests without lateral expansion of the soil constituted 100%

Figs. 1b and 1c list the test results.



Figs. 1b and 1c.

b- diagrams of the dependence of the errors of various earth pressure cells on the value of vertical pressure with the possibility of lateral expansion of soil and its relative radial strain being equal to 2.10^{-2}.

c- the same for cells type PDGS-3

1- shell; 2- cover, 3- reference pressure gauge, 4- connecting pipe, 5- rubber diaphragm, 6- soil, 7- cell under investigation, 8- miniature cells type MCM-2, 9- porous compressible material, 10 at a relative strain of soil in the radial direction, equalling 1.10^{-2}, 11- at a relative radial strain constituting 2.10^{-2}.

The data on the considerable concentration of stresses in the cells in the case of uniaxial compression and the possibility of lateral expansion of the soil are confirmed only partially. Only in diaphragm cells (with central and annular membranes) did we observe an appreciable increase of the output signal (up to 45%).

Cells with a hydraulic multiplicator react to a decrease of lateral pressure much less noticeably.

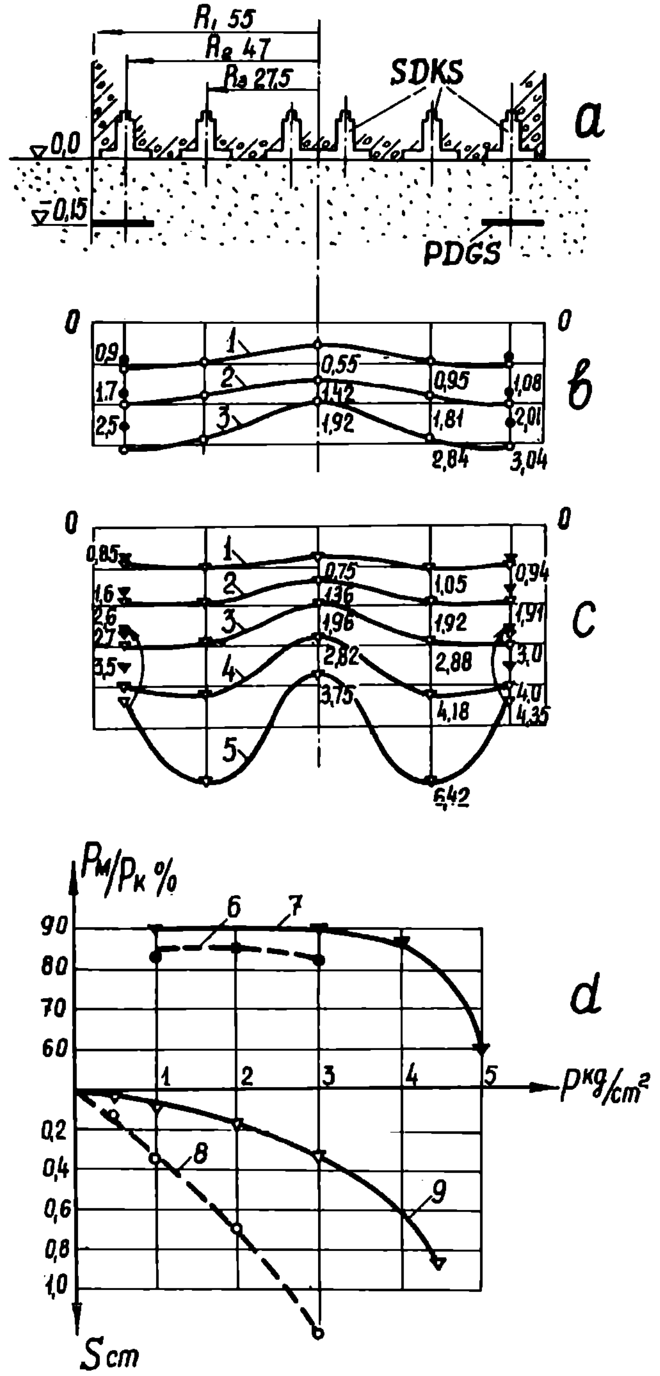


Fig.2. Location of stressmeters in round footing and earth pressure cells in soil under a footing, as well as pressure and settlement diagrams obtained in tests carried out in a large tank

a- schematic diametral section of footing and soil basement with stressmeters and earth pressure cells mounted;

b- charts of distribution of pressures for first loading,

c- the same for second loading,

d-diagrams of relative stresses and settlement of footing

1- under conditions of an uniformly-distributed load on tho footing, equal to 1 kg/cm²; 2,3,4 and 5- the same at loads of 2,3,4 and 5 kg/cm² respectively; 6- ratio of readings of cells located in a soil mass and stressmeters located in a footing at first loading; 7- the same at second loading; 8- footing settlement at first loading; 9- the same at second loading.

The procedure of carrying out check tests in a large soil testing tank is made clear in Fig.2 a. Fifteen contact stressmeters for soil type SDKS /1/, /2/ are mounted in the toe of the reinforced concrete footing. Five flat cells type PDGS-Ib /1/ located in a zone where as a result of the footing being loaded the forcing out of soil and a consequent decrease of lateral pressure occur are placed under the footing.

Starting with pressures of 3 and 4 kg/cm² during the first and second loading, respectively, owing to the formation of displacement zones in the plane of location of the earth pressure cells, the values of the ordinates obtained according to the cells (black dots) begin to fall behind the values measured by the contact stressmeters (white dots). The side ordinates of the chart of distribution decrease, which is recorded both by the contact stressmeters and, particularly perceptibly, by the cells (Fig.2b,c,and d). The formation of displacement zones is confirmed by the steepness of the footing settlement diagrams (lines 8 and 9 in Fig.2,d).

Analysing the results of the tests in the tank, we conclude that, regardless of the reduction of lateral pressures in displacement zone under the footing, the sensibility of cells located in this zone does not increase. On the contrary they record a reduction of stresses in this region.

Therefore, the check tests in the laboratory setup and in the tank showed that flat rigid cells do not produce the considerable errors.

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Chairman Stanley D.Wilson

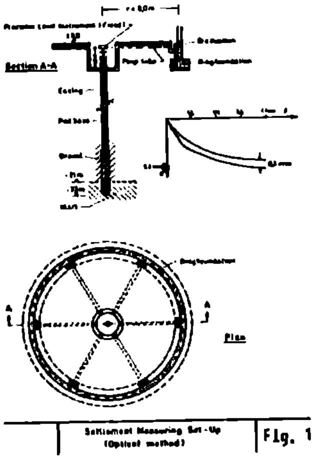
Thank you Mr.Lasebnik for your contribution. The next will be Mr.Gruber, FRG.

Dipl.Ing.Norbert Gruber /FRG/

We take this opportunity to present the various methods and equipment used for the last few years for measuring settlements of foundations in southern part of Germany.

1. Optical Method

Figure 1 shows the optical method. The point of reference - with respect to which settlements are measured - is anchored at a depth where no measurable settlements are expected. At this depth a rod-base was cemented in. On top of this base a precision level was fixed, which remained in place for whole of the observation period.



Settlement Measuring Set-Up (Optical method) Fig. 1

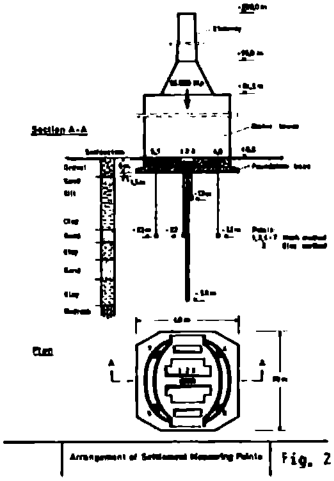
The settlement readings are made on graduated staffs fixed in the ringfoundation. The accuracy of the readings is 0.05mm. Such a set-up is advantageous in observing differential settlements especially for ringfoundations. The device shown in Fig.1 was used for observing settlements of a foundation for a large rotating areal. The specifications demanded that the differential settlement of the construction was not to exceed ±1mm. The observed total settlement was 2.8 mm and the differential settlement 0.6 mm.

II. Electrical Method

Figure 2 shows a chimney and boiler tower of a power plant constructed on the banks of river Danube. The total weight of the plant is as great as the total weight of the Moscow Television Tower, that is 55000 metric tons.

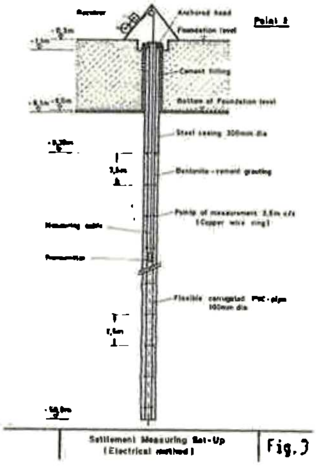
The purpose of the measurements was

a) to measure compression of each layer below and at the center of the foundation up to a depth of 60m (points 1-3).
b) to measure total compression below the edges of the foundation as a result of wind loads (points 4-7)



Arrangement of Settlement Measuring Points Fig. 2

For the first case a flexible corrugated PVC-pipe with copper rings at a distance of 2.5m center to center was used (Fig.3)



Settlement Measuring Set-Up (Electrical method) Fig.3

The pipe is connected to the soil with the help of cement-bentonite grouting. A transmitter is lowered in the pipe and a change in frequency is signalled in the receiver every time the transmitter passes the copper rings.

The accuracy of this set-up (System Idel) is of the order of a few mm which primilary stems from the mechanical parts of the set-up.

It is printed out here, that the used corrugated PVC-pipe shown in Fig.4 can take up compressibility only up to 20-30% and because of it being brittle the pipe is prone to damage. For that reason such a pipe is not suitable for installation in deep bore holes for measurements at greater depths and where large settlements are to be measured.

A more suitable flexible pipe as shown in Fig.5 and 6 is made of plastics covered steel wire spiral around which synthetic foil is pasted on. Till now it was feared that such a pipe will act as Faraday's screen thus preventing measurements of change in frequen-

Fig. 4 Flexible corrugated PVC-pipe (100 mm dia.) with copper ring

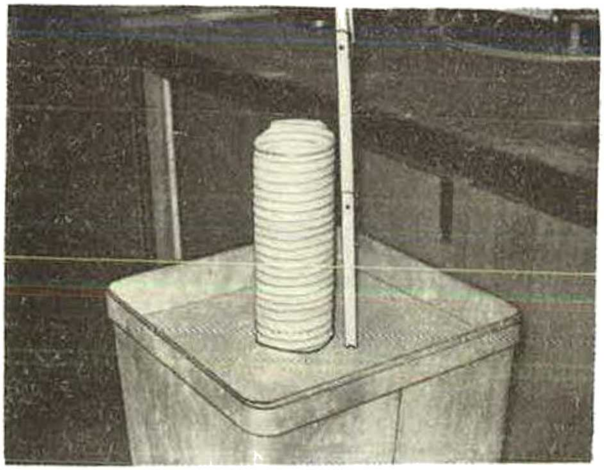


Fig.5. Flexible pipe with plastics covered steel-wire spiral and copper rings

Fig.6. Flexible pipe with plastics covered steel-wire spiral and copper rings-rate of compressibility.

cy when the transmitter passes the copper rings fixed outside of the pipe. But with experiments such an effect was not noticed because of the fact that the two ends of the steel-wire spiral are not connected together.

The experience has shown that, where large settlements are to be measured and where pipes resistant to special strains are to be installed in bore holes, a flexible pipe shown in Fig.5. and 6 combined with the Electrical Method (System Idel) is most effective.

III. Mechanical Method

For comparison a simple mechanical method shown in Fig.7 was adopted. This was done with the help of invar wire anchored at the bottom of the bore hole and kept in stretched condition by weight suspended at the other end. The invar wire is protected by a two inch PVC-pipe. The amount of relative movement of weight with respect to foundation level equals the compression of the soil layer for the length of the wire up to the anchored end. The movement of weights is measured with an accuracy of one over one hundred mm.

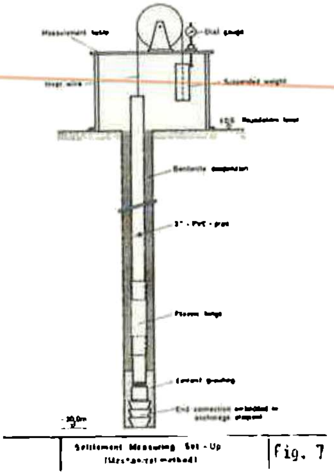


Fig. 7

For the observation of settlement due to wind loads automatic drums with record sheet instead of dial gauges were installed at the four edges of the foundation base. This way the settlement due to dynamic loads is to be measured for the top thirty meters. Automatic recording takes place when the wind velocity exceeds a predetermined value.

Chairman Mr.Stanley D.Wilson.

Thank you Mr.Gruber for your report.
The next will be Mr.Perlea from Romania.
Mr. Perlea will you please.

Mr.Vlad D.Perlea (Romania)

Monsieur le President a suggere une discussion sur le probleme de la precision des mesures de tassement dans les massifs en terre et sur les moyens d'augmentation de cette precision.

Un dispositif de mesurage des tassements en profondeur est l'appareil electro-inductif, dont le principe d'utilisation est montre dans la figure 1; en deplaçant le traducteur par l'interieur d'un anneau metalique, solidarise avec le terrain a un niveau donne, on determine la position de l'anneau correspondant a la valeur maximum de l'intensite du courant d'induction magnetique.

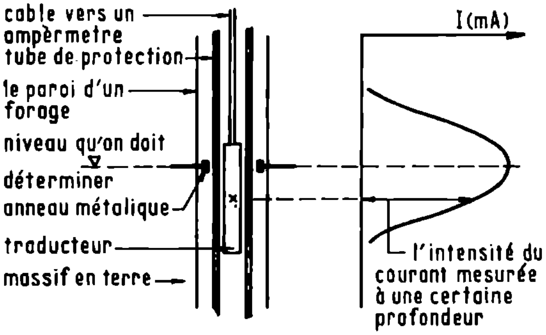


Fig. 1. La principe de la methode electro-inductive

On ne peut faire la determination du pic de la courbe de variation de l'intensite du courant qu'a une precision d'environ 5 mm.

La precision des mesures peut etre sensiblement amelioree en utilisant un couple de traducteurs solidarises a une distance determinee, lies du point de vue electrique de sortes que les intensites des courants d'induction magneitique se soustraient (fig.2)

C'est ainsi qu'au lieu de chercher le maximum d'intensite, on doit determiner la position du couple de traducteurs lorsuq l'intensite est nulle ; on constate qu'a proximate du point de nul la variation de l'intensite est tres nette, donc la precision des mesures est bien accrue.

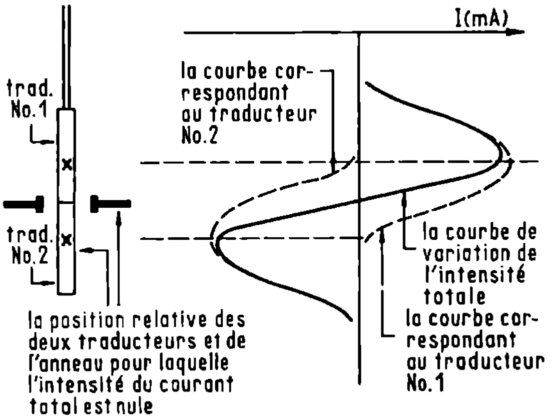


Fig.2. Le moyen propose pour l'amelioration de la precision des mesurements (pour la demonstration du principe on a considere le couple des traducteurs fixe et l'anneau mobile).

Chairman Mr.Stanley D.Wilson
Thank you Mr.Perlea.And the last in our discussion will be Dr.Tcheng. Dr. Tcheng, will you please.

Dr.Y.Tcheng /France/

We have just been listening to a certain number of lecturers who have presented the various techniques of stress measurements.

We intend to point out the interest of these measurements in practice.

Effectively,while testing the passive earth pressure,we have measured from behing a retaining wall,the stress down the soil and the deformations in surface.

The results of this survey are summed up in the following diagrams.

Figure No.1 gives the value of the vertical stresses recorded by the captor have appeared well ahead the failure of the soil.

Effectively, Δl/h rises over 3% at failure while, the captors disposed inside the dislocated mass of earth show a maximum value as soon as Δl/h reaches 2% (Δl is the displacement of the wall and h it's height)

2-Therefore,it is valuable to notice that

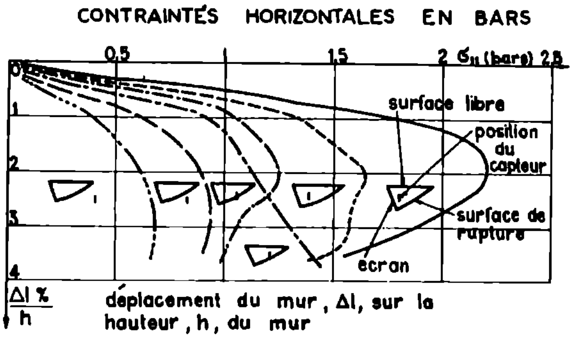


Fig.1

the captors embedded down the soil for measurements may help forewarn the danger of failure well ahead devaster.

The surface deformation measurements have come up to the same results but we cannot give them in this paper for luck of place.

The figure No.2 gives the valuc of tho stresses recorded by the captors disposed along a single horizontal alignement originating from the wall.

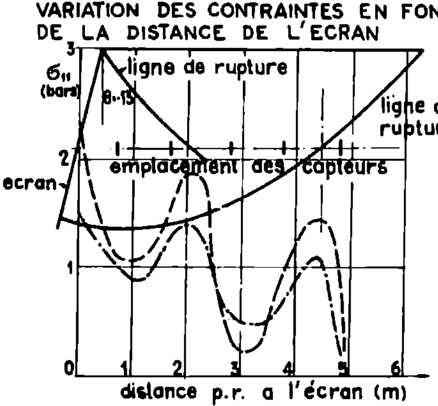


Fig. 2

The exact location of the two peaks on the curve corresponds to the deformation of the lines of rupture.

Thus, we have ascertained that measurements on the site could forewarn in time of the rupture of the soil and even locate the failure lines.

Chairman Mr. Stanley D. Wilson

Thank you Dr. Tcheng for your discussion. Now at the end of the work of our Session I want to say some words.

Prof. Stanley D. Wilson (USA)

In my opening remarks I pointed out that there existed a variety of excellent devices for measuring settlement and heave foundations. Most of the discussion this afternoon was connected with settlement devices and it is evident not only that improvements to present devices are being made, but also that many new devices are still being developed. I think the reason for this interest is that settlement is the most important measurement that needs to be made with respect to all structures. Differential settlements produce cracks and high stresses, and the ability of the structure to resist these stresses depends not only on the magnitude of the settlements but also upon the rate with which they develop. Therefore, great precision is needed in most cases to pick up the rate of differential settlement at an early date.

In my opening remarks, I stated that there were many unsolved problems relating to pressure cells. Therefore, I was very pleased to note the interest shown in these problems by many of the speakers. However, most of the attention was focused on the design of the cell itself. In the opinion of the chairman, however, the greatest deficiency is in the method of installation, and it is in this area that much remains to be done. I understand that this specific problem is being studied intensively here in the USSR at the Research Institute for Bases and Underground Structures, and I hope that some of their results will be published in the near future.

In conclusion, I would like to express my thanks to the speakers this afternoon and to our hosts of the USSR in sponsoring this session. My vice-chairman Mr. Baranov, and the recording secretary Mr. Mamonov have been most helpful in arranging the details of this session. Their keen interest in instrumentation and their fruitful work in progress at the Institute will certainly lead to important publications in the future.

I now close this first technical Session of the VIII International Congress.

WRITTEN CONTRIBUTIONS

STRESS MEASUREMENT METHOD, EFFECT OF THE SAND BASE FORMATION TECHNIQUE.

D.S.Baranov, S.V.Downarowicz, D.E.Polshin, V.F.Sidortchuk (USSR)

Effect of the footing configuration and the sand base formation technique upon stresses and settlements of the base was experimentally studied on circular load plates I50 cm in diameter and rectangular load plates 60x 300 cm. All the load plates had on equal area and were not deepened into the soil. The tests were carried out in special boxes 8x8x8 m in which the 4.5 m deep sand bases were prepared. The sand was of medium coarseness with 67% of 0.5-0.I mm fraction and moisture content W within 3.7 and 4.2%. Although the bases were formed by two different techniques, the unit weight of the sand was kept constant in all of the tests. By the first method a test box was filled by means of a clamshell bucket from the constant height H = 4 m above the sand surface. The unit weight was checked for each layer of sand and was equal in average to I.65 t/m^3 On the second method the sand was placed by letting in fall from the height H = 0.5 m with subsequent vibrator compacting of the layers from initial unit weight of I.5I up to I.65 t/m^3.

Stresses in the soil and on bottom surface of the load plate were measured by means of pressure sells with hydraulic transformers. The device were 70 mm in diameter and I0.5 mm high and had their equivalent modulus of E = (2 + I0) x I0^4 kg/cm^2. Previosly the pressure sells had been thoroughly tested /I/ and as a result the values were found of sistematic and random errors caused respectively by soil stress concentrations within pressure sell location area and non-homogeneity of the soil.

Curves on a Fig.I confirm earlier obtained by Downarowicz data showing that the settlements of oblong load plates, are slightly less than those of more compactly shaped - square or circular - plates of the same area. It is also seen from the Fig.I, that the deformations of the base depend not only on the porosity of the sand but on the technique of its placing.

Initial stress state of the base produced by its dead weight depends on placing technique either. Thus, with the free filling through the height of 4 m the ratio of side pressure was of ξ = 0.42 ± 0.I3, while with the vibrator compaction it was of ξ = I.I6 ± 0.58.

Diagrams of stress σ_z produced by external load /I/ show that σ_z is also affected by the base placing technique. Influence of the footing shape is noticeable only down to 3 m, where σ_z is greater for the circular load plate.

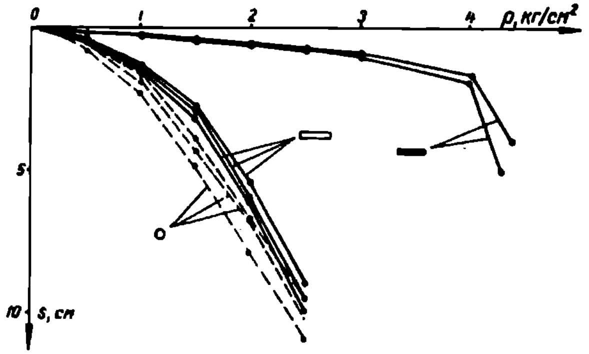


Fig.I. Relationship between the load plate settlement S and average pressure P at the bottom surface of the load plate.

▬ - rectangular load plate 60x300 cm on the sand base compacted layer by layer,

O, ▭ - circular I50 cm dia. and rectangular 60x300 cm load plates on the sand base filled through the height of 4 m.

As it is seen from Fig.2 a distributation of the contact pressures is close to uniform. At the values of P beyond the certain level a transformation of the stress distribution was observed. For the rectangular load plates its begining coincided with the begining of an intensive extrasion of sand surface.

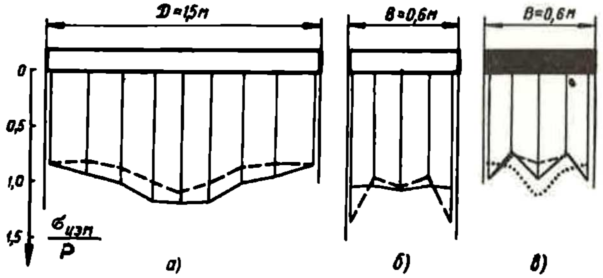


Fig.2. Contact stress $\sigma_{изм.}$ diagrams. a,b - for the sand placed through the height of 4 m, c - for the sand compacted layer by layer.

--- for P = I kg/cm^2, —— for P = 2 kg/cm^2, ········ for P = 4 kg/cm^2.

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A SIMPLE BOREHOLE EXTENSOMETER FOR THE OBSERVATION OF SETTLEMENT OF BASES

J.B. Burland (United Kingdom).

The application of load to the ground by a structure provides an opportunity for obtaining values for the in-situ properties of the ground, in particular its compressibility and its response to loading with time. The measurement of these properties in the laboratory and their application in practice are notoriously unreliable.

Provided the strata are sufficiently well defined and the structure is not too complex the effort spent on taking detailed measurements of settlement can provide information which will be of value in the design of future structures on similar geological formations.

Even when the structure is particularly simple in plan and the ground conditions reasonably straight forward the interpretation of settlement observations is difficult. This is because both the compressibility of the ground and the stresses induced by the foundation vary with depth. The value of settlement observations are greatly enhanced if the compression of various descrete layers beneath the foundation are also measured. An example of the value of such detailed measurements in evaluating the compressibility of the ground at various depths is given in a paper to this Conference by Burland, Sills and Gibson (1973).

As part of its research programme into ground movement the Building Research Station has recently depeloped a simple and accurate method of measuring relative displacements along a borehole which should prove very useful for settlement studies (Burland, Moore, and Smith 1972). The system has two basic components: a permament circular magnet axially magnetised which acts as a marker embedded in the ground and a reed switch which is used as a sensor. As the reed switch moves axially into the central field of the magnet it snaps closed thereby activating an indicator light or buzzer. This system appears to be well suited for incorporation in field measuring equipment since it is simple, reliable and accurate and is very cheap.

A number of magnets may be installed along a borehole. A central guide tube runs down the centre of the borehole and the position of each magnet can be determined by means of a sensor (containing a reed switch) attached to a steel tape. This sensing system gives an accuracy of about 0.5mm. A much more accurate system is to install a permanent datum rod down the guide tube with a reed switch mounted opposite each magnet. Only small axial movements of the measuring rod are required to sense each magnet and these can be measured at the surface by a screw micrometer. Accuracies of the order of 0.05 mm can be achieved with this system. By installing magnets at various depths down a borehole beneath a foundation not only can the settlements at various levels beneath a foundation be accurately determined but if the bottom magnet is embedded in a firm stratum the installation can also be used as a deep bench mark. By installing pore pressure gauges beneath the foundation as well, the in-situ values of both the compressibility and the coefficient of consolidation of each layer of soil can be evaluated. It is only by making such direct measurements that the accuracy of laboratory methods can be reliably assessed.

Finally, it is worth noting that the system can also be used to determine the heave occurring at various depths beneath excavations. The widely used method of grouting a sleeved rod into the bottom of a borehole and levelling to the top is very vulnerable. With the system described here any magnets located in the ground beneath the excavation will be safe provided access down the guide tube is maintained.

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STRESSES AND SETTLEMENTS MEASURED UNDER RAFT FOUNDATIONS OF MULTI-STOREY BUILDINGS.

C.M. Gerrard, M. Kurzeme (Australia).

The CSIRO Division of Applied Geomechanics is currently conducting a study of the performance of raft foundations of five multi-storey buildings in Perth, Australia. The study integrates the interacting phases of the measurement of material properties, prediction of the performance of the rafts and field observation of this performance with the aim of defining methods of improved design and analysis. This report is concerned with the field observation phase of the broader integrated study (Gerrard *et al*, 1971).

The area on which buildings are sited is a Quaternary alluvial terrace subsequently covered by sand of aeolian origin. The total thickness of the desposits over a calcareous shale bed rock is about 30 m, and the water table is about 8 m below the natural surface. The soil profile consists of 5-9 m of the aeolian sand in the top layer,followed by 4-10 m of stiff silty clay, with the remainder being an extremely variable interbedded system of sands, silts and clays.

The five buildings range from 13 to 30 stories high, including one or two basement floors. They are of steel and reinforced concrete frame construction and all contain some stiffening walls. The reinforced concrete foundation rafts vary in size from 30 m x 30 m to 50 m x 60 m with thicknesses between 1.3 m and 2.5 m. Design estimates of the average gross contact pressures for the five buildings range from 210 KPa to 350 KPa, the net pressures being less than these because of the excavations for the basements.

The main problem in the design of these rafts was the interaction between the settlement behaviour of the layered deposits and flexural behaviour of the rafts as stiffened by the building structures. Considerations of the total settlement and the effect of the limited seismicity of the area were secondary.

The type and distribution of instruments installed to measure the performance of the foundations were oriented towards the main design problem. Vertical settlements were measured at the raft surface and at soil layer interfaces together with the pore pressures, occurring within layers, and the lateral movements down the profile. The bending of the rafts was assessed by measuring contact pressure distributions, column loads, and deformed shapes.

The numbers and types of instruments installed are summarised in Table I.

In addition, column loads were measured in fifteen columns on four of the buildings, while within one of the foundation rafts three arrays of concrete stress and strain measuring devices were placed.

The observed raft performance, as indicated by the instruments, has given some confirmation of the expected trends of behaviour. The earth pressure cell readings increased in proportion to the total load applied as determined by the stage of completion of the buildings.

TABLE I

Response Parameters	Type of Instrument	No.of Foundations Instrumented	Av.No.of Instruments per Instrumented Foundation
Gross Contact Pressure (Earth Pressure Cell)	170 mm dia.Carlson type, and 450 mm dia. oil filled with bonded strain gauge pressure transducer	5	18
Pore Pressure (Piezometer)	Casagrande type, and Carlson type	5	6
Deformed shape (Precise levelling)	Surface mounted stainless steel plugs	4	30
Vertical movement in profile (Precise levelling)	Anchored plates with isolated vertical rods	3	9
Lateral movement in profile	Pendulum activated bonded strain gauge transducer	1	2

The general distribution of contact pressures under the raft indicated concentrations of pressure near the raft edges of up to twice the average values. The piezometers indicated that excess pore pressures were negligible. This immediate dissipation was expected because of the very short drainage paths resulting from the interbedded system of layers. The vertical settlement through the profile showed a tendency towards constant vertical strain. A trend in this direction, although not quite so marked, was expected on the basis of applying simple elastic theory to problems involving rigid bases at finite depth. Although the deformed shape of the rafts was generally concave upwards with minimum settlements at the corners, a non-linear response to increasing load was indicated by changing patterns of deformation. For example, on one foundation raft at low load there was a tendency to axial symmetry about the centre point whilst at higher load the symmetry tended to be about the vertical plane through one of the raft axes.

The construction period for the five buildings will extend over five years. To date three buildings are complete and one is only at foundation level. The measurements to date indicate satisfactory functioning of the instruments giving data that will be essential to the broader integrated study.

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REINFORCEMENT OF FOUNDATIONS OF THE ST. FRANCESCO BASILICA IN ASSISI AND CHECKING ITS RESULTS WITH PHOTOCLINOGRAPHS.

R. Diamanti - Emma Diamanti Derossi (Italy)

The Basilica of St. Francesco in Assisi is one of the most important gothic monuments in Italy, not only because of its architecture but also because it contains frescos by two of the greatest painters of the time: Giotto and Cimabue.

Construction of the Basilica began in 1228, and during the centuries continuous strengthening and repair works were required.

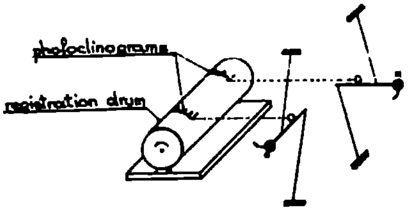
In spite of the repairs, in 1960 there were widespread capillary cracks and also macroscopic cracks in the walls and floor of the Upper Church. Analysis of the cracks showed an increasing movement of a rotation of the whole facade towards the front square.

First, an accurate geological survey of the foundation ground was carried out, and the condition of the foundation walls was verified. Thereafter the strengthening began.

The Church is built on a spur of Mount Subasio. The geological strata, which date between the inferior Lias and the Eocene, are conform and change with continuity from limestone to marly-limestone and sandy limestone. The Juro-Liassic strata can be considered the central core of the Subasio's anticlinale, above it, there are the cretaceous softer limestone. The Church's foundations lay on this last formation, which appears in two different and distinct ways: "rosy limestone" is small thickness strata, then "rosy scaglia" representing the superficial and degraded side of the formation. The geological survey showed the existence of numerous open fissures and a great release of the strata, with subvertical cavities which were sometimes very big.

The strenghtening consisted of a series of cement-water injections. The grouting was carried out through vertical and raked boring around and through the foundation masonery itself. 153 holes, totaling about 2500 m. were drilled, and 1900 t. of cement was injected.

Photoclinographs, highly sensitive registrating equipment, used also for controlling movements of dams, were used to check the effinciency of the grouting.

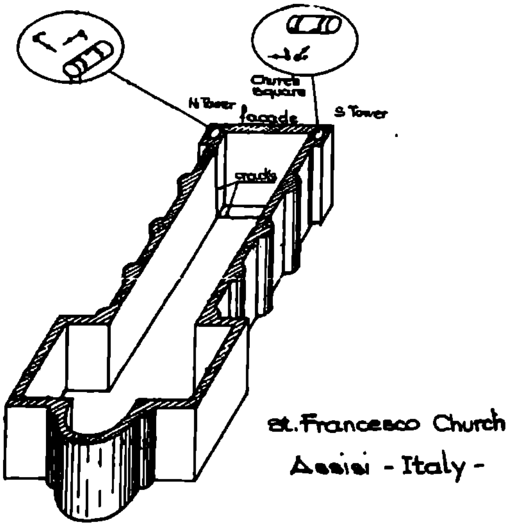


PHOTOCLINOGRAPH SCHEME

The photoclinographs are bifilar suspended horizontal pendulums with a very high sensitivity, function of period, which can be fixed, as required, by varying the inclination of the axis of the pendulum; thus registering even micromovements of the ground and buildings upon which they have been installed. They are used in pairs and arranged orthogonally in order to obtain, through the composition of the two orthogonal movements, the actual movement of the construction of which they are a solid part. Each registration of the photoclinograph projected on photosensitive paper (photoclinogram), can include for example a period of one month. By developing the differential equation of the photoclinograph movement, from the photoclinogram variations, we obtain the micromovements of the construction.

The results can be given in arc seconds or radiants or in diagram.

The group of photoclinographs were placed in the two towers alongside the facade of the Upper Church.



UPPER CHURCH ASSONOMETRIC SCHEME

Recording began in February 1961. The relative photoclinograms showed variations of inclination in the facade towards the square in front of the Church and to the right.

Owing to the grouting work the movement noticeably decreased untill it disappeared completely after 3 months. By the end of the control, about two years later, the recordings made showed no angular movement of the facade connected to the settlement of the building.

HORIZONTAL DYNAMIC STRAIN MEASUREMENTS FOR EMBANKMENTS.

Rudy J. Dietrich and J. Ronald Salley (U.S.A.)

General

The United States Department of Transportation (DOT) and the Atchison, Topeka, and Santa Fe Railway Company (ATSF) are jointly sponsoring an investigation into methods of providing more stable railroad track structures for high speed, heavily loaded trains. A test embankment has been established adjacent to the main line of the ATSF system for this investigation. The test embankment is nearly two miles long and is comprised of a minimum thickness of six feet of compacted, highly plastic clay. The embankment is divided into nine test sections, each with a unique track support system. The support systems include concrete ties at four different spacings, a concrete slab, a continuous concrete beam, a precast beam, stabilized ballast, and a control section of the standard ATSF design. Each test section has identical embankment instrumentation.

Embankment Instrumentation

The response and performance of the track structures will be influenced by subgrade performance. A system of instrumentation was developed to observe the embankment behavior in order to assist in the evaluation of track structure performance. Emphasis was placed on reliable measurements of dynamic embankment strain. Measurement of long term deformations and volume changes in the highly plastic clay embankment was also provided.

Each of the nine test sections has one principal instrument array supplemented by additional instruments spaced to verify that the performance at the main arrays is typical for that test section. The main array instrumentation includes vertical extensometers, portable horizontal extensometers which will be inserted in flexible horizontal tubing embedded in the embankment, pressure cells, and moisture-temperature cells. The instrumentation was specifically designed for this project. A special consideration was to eliminate the requirement for operations from the top of the embankment which would interfere with rail traffic. The portable horizontal extensometers are considered particularly unique and are discussed in this paper.

The use of accelerometer sensors and a magnetic tape data acquisition system was considered but was eliminated because of the high capital cost of instrumentation and recording equipment. Consequently, it was decided to adopt a system utilizing LVDT transducers for sensing permanent and transient deformations. This equipment will measure displacements directly, and recording equipment will consist of digital readout and chart recorders.

Horizontal Extensometers

At each main instrument array, horizontal four-inch diameter, relatively flexible, corrugated polyethelene tubing was placed in the embankment at four levels during construction as shown in Fig. 1. The tubing has relatively rigid PVC couplings at 2.5 and five-foot spacings which are anchored in the embankment. Strain rods with a hooking device to engage the anchored couplings are used to measure the static horizontal deformation of the embankment relative to the end of the tubing.

Dynamic horizontal deformations in the tubings are measured with portable extensometers with gage lengths of 2.5 and 5.0 feet, see Fig. 2. Hydraulically-operated anchor shoes (pistons) at the ends of the extensometers extend and lock into the tubing couplings. An LVDT at one end of the extensometer records dynamic deformations caused by passing rail traffic. Three portable extensometers were constructed for the project. They can all be temporarily installed in the same casing or in any of the four casings in one main instrument array. Recordings of embankment strain at many locations during initial testing will serve as the basis for selecting a few locations of particular significance for subsequent strain measurements.

The horizontal tubing also provides openings in the embankment which would be available for insertion of other instrumentation, if desired. It would also be possible to insert a portable accelerometer package into the interior of the embankment for displacement measurements in three directions.

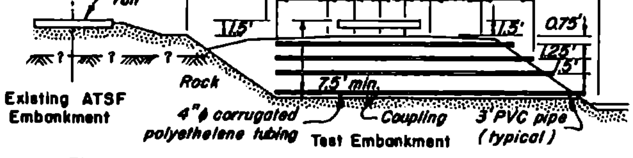


FIG. 1 HORIZONTAL TUBING - MAIN ARRAY

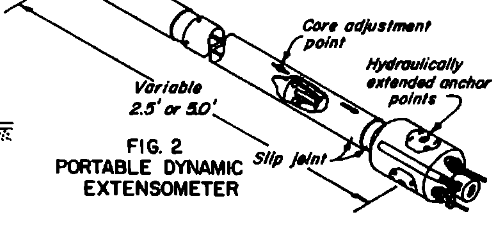


FIG. 2 PORTABLE DYNAMIC EXTENSOMETER

AN OPTICAL METHOD FOR MEASURING THE DEFORMATIONS OF CONCRETE DIAPHRAGM WALLS AND SIMILAR STRUCTURES. V. Escario (Spain)

Slurry trench concrete diaphragm walls, due to their high stiffness, may require special procedures to measure their small deformations. The principle of the method we have developed is the following: a vertical pipe with longitudinal grooves is cast in the concrete of the wall to follow the deformations. A cylinder provided with four wheels is lowered along the longitudinal grooves; a regular flashlight is located inside the cylinder and, on top of it, a target formed by a glass with circles drawn every 1 mm is illuminated (fig. 1). With an optical plumb (fig. 2) placed on its tripod above the pipe, the lateral displacements suffered by the calibrated glass as it is moved along the whole depht are easily determined.

Fig. 1. - Wheel guided iluminated target

As it is the first prototype, instead of constructing a separate guided cylinder, we have adapted it to the top part of a comercial slope indicator. It is raised and lowered by two lateral nylon strings so that they do not interfere the visibility of the target. The depth is measured with a graduated steel tape which is fixed at one side of the cylinder. The inside diameter of our pipe is 7,5 cm, corresponding to the slope indicator used. The cylinder is 63 mm outside diameter and 160 mm long. The best results for a clear reading of the graduated glass were obtained by having the divisions and figures transparent in a shaded field with properly diffused light.

The optical plumb is of a commercial high precision type, directly adaptable to the head of the level. Actually it does not define a vertical line, but a vertical plane and that is why the graduations in our glass are circles, where the tangents projected by the plumb give the readings; but a set of parallel lines might also be used. By placing the level in two perpendicular directions the displacements in both of them are obtained (for piles for instance).

The maximum deflection which can be measured is equal to the inside diameter of the casing used. Actually it is a little less, since the graduated glass is somewhat smaller in diameter. If the deflections expected are large, a larger diameter pipe might be used. In order to take full advantage of the available range, the casing should be placed perfectly vertical, what can be attained in several ways. If the level is provided with stadimetric lines, they may be used as reference lines instead of the vertical, what considerably increases the range of measurement.

The accuracy of our optical plumb is of about 1" (0,25 mm in 50 m) and the amplification of 32 times; nevertheless there are other factors which reduce this high precision, as for instance the play of the wheels. In the tests carried out up to this moment with concrete diaphragms of about 15 m depth, the maximum differences obtained when repeating several readings were under 1 mm and very often of less than 0,25 mm. A correction may be necessary in some cases because the target is not in the middle of the four wheels.

This method may also be applied in somewhat different ways, as by placing fixed points in the pipe, conveniently iluminated. For instance S. Uriel is using as targets the filaments of computer type minibulbs.

Acknowledgments. Several persons have collaborated in this project but specially E. Castillo Ron, J.A. Sahelices, J.L. Martín and L. Vela.

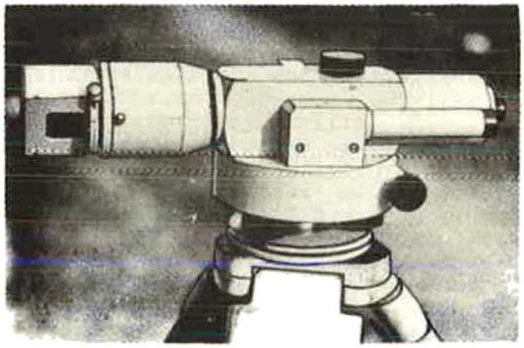


Fig. 2. - Optical plumb used

FOUNDATION INSTRUMENTATION FOR THE NATIONAL WESTMINSTER BANK TOWER.

E.T. Haws, R. Tabb
(Great Britain)

The 183m high National Westminster Bank tower will be supported by a total of 370 No. 1.22m dia. under reamed bored piles 26.5m long, constructed completely within the London Clay. The pile cap consists of a duodecagonal reinforced concrete raft 54m equivalent dia. and up to 4.5m total thickness.

Instrumentation sponsored by the Construction Industry Research and Information Association (CIRIA) and the Building Research Station (BRS), has been installed to investigate the pile/raft loading distribution and settlement profile under both short and long term loadings. Four pile load cells and twenty Gloetzl total pressure cells are installed approximately along the radius of prevailing winds. Results from this instrumentation should lead to greater design confidence and economies for large piled foundations.

The 1.2m dia. 8,000 kN capacity pile load cells are positioned near the tops of their pile shafts. Within each cell are located eight photoelastic load columns spaced evenly around an inner perimeter, through which all the 8,000 kN load is transmitted. Reading is by lowering an endoscope, light source and polarizer down the 150mm dia. central access tube. The light source and polarizer are positioned behind each load column in turn using a hinged rod attached to the endoscope. The fringes are read through an analyser at the endoscope eyepiece at basement level, some 8m above the cell.

A 'backup' measuring system is incorporated in the cell design and consists of recoverable displacement transducers measuring cell compression at two diametrically opposed positions. The resolution of this 'backup' system is better than 5% of the full, evenly distributed load range, but should large loading eccentricities occur, resolution is lower.

The Gloetzl total pressure cells are installed at the raft/clay interface and covered with the high strength raft concrete. Five have surface temperature sensors to assess the effects of heat of hydration within the massive concrete raft. The twin hydraulic leads and temperature sensor cables run up to an instrument house located just outside the foundation area. To obtain a better understanding of how the Gloetzl cell's measured pressure relates to applied load under differing installation conditions, a Finite Element study was undertaken. Results indicated that for the proposed installation an overread of several percent could occur. Experimental verification of this estimate is being undertaken by placing a mock-up of the proposed installation inside a 900mm dia. pressure cell, the test being performed under pore pressure dissipation conditions.

The foundation's response to dynamic rocking will be recorded by a displacement transducer mounted on a deep datum anchored 50m below raft level into the very dense Thanet Sand. It is located at the outer perimeter of the raft along the direction of prevailing winds, and with the transducer attachment angular rotations can be interpreted with better accuracy than could be obtained using commercially available tiltmeters.

The BRS have installed a settlement gauge recently designed by them. The gauge, anchored 64m below the underside of the raft and 6m into the chalk, contains magnets mounted in plastic holders which are sprung into the sides of a 150mm dia. drill hole. Grout, with a modulus matched to that of the surrounding in-situ strata, is pumped down a central grout tube thus completing the installation. Under loading the gauge lengths compress and magnet positions are detected using reed switches mounted on a stainless steel datum bar sliding within a central guide tube, (Burland, Moore and Smith 1972).

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METROLOGY AND TECHNIQUE OF STRESS MEASUREMENT IN A SOIL MASS. V.Kheifits (USSR).

In I967-I97I the Scientific Research Centre of Hydroproject conducted a number of studies dedicated to creation of measurement means and to improvement of procedure and technique of stress measurement in a soil mass of high dam cores I . They resulted in the following:

I. Development of metrological methods for testing of soil-stress detectors (SSD) 2 based on their interaction with soil mass at stressed states and stressed paths which exceed all possible range of their future service conditions.

2. Introduction of new test devices enabling simulation, with a specified error, of any predetermined conditions of uniform stressed state on the boundaries of the soil sample containing the detector within (USSR patents NN. 247577, 286307, 296970). The specific feature of these devices is the design of their cell walls concurrently deforming with the soil sample. Due to this shear stresses between the cell walls and the soil are absent, and the principal stresses planes conditions on their contact are realized. The cell providing for simulation of one-dimensional strain conditions was developed contemporaneously with and independently of the similar annular cell of E.Fumigalli 3 , it comprises a set of rigid and non-rigid rings placed between the direct-stress machine plates. To prevent the shear stresses appearing on the sample ends due to some insignificant radial deformations of the ring walls, thin rubber discs are introduced between the soil sample and end plates. They are penetrated by channels filled with low-compressibility fluid. For creation of any axis-symmetrical stressed state, the annular cell has a rubber envelope separating the soil from the side walls. The cell of an apparatus used for creating in the soil sample the three-dimensional stressed state with 3 independent principal stresses has the shape of a cube constructed of deformable walls which are compressed via rigid plates by jacks in three mutually perpendicular directions.

3. Performance of metrological testing of different types of SSD showing that errors of tens of per cent occur when the data of the detector error obtained at one stress-strained state is applied to other condition of the detector-soil interaction. Metrological tests of rigid thin disc-type detectors (the thickness to diameter ratio 0.2 and the reduced modulus of deformation 3.10^9 Pa) revealed that their output signals depend on:

- soil deformations developed in detector plane (the formation of "associated rigid soil nuclei" due to friction causes stress concentration from +80 to -50%);
- stress gradient (when the sensing element is asymmetric relative to the middle plane). So the sign of its error depends upon the direction of the detector plane normal in relation to the direction of the radius of curvature of the deformed section and the value of error depends on the radius of curvature, and it amounts to several tens of per cent (the so called "effect of orientation").

4. Evaluation of dependance of coefficient of variation of measured stress values upon the nonuniformity characteristics of the soil deformation properties i.e. coefficient of variation of its modulus or density , grain composition and detector size . In this case at radius of stress correlation , value will be:

(2)

where = Laplace's integral.
For loose soil and its maximum uniform placement radius of stress correlation is proportional to the mean size of particles and is 5-I0 times larger.

5. Findings to the effect that for ensuring the detectors accuracy, in case of measuring stresses in nonhomogeneous soils in three-dimensional stress-strained state, it is not sufficient to satisfy the requirements for the maximum stiffness at the minimum thickness which were earlier imposed upon the SSD. According to the new specifications the detector should have minimum volume compressibility, minimum thickness, the ability to alter its shape (to extend and to bend) together with the soil and - for the cases of stress measurement on nonhomogeneous soils - its area should be sufficiently large.

On the base of these specifications a detector has been constructed (USSR patent N. 30I584) which comprises a thin rubber sheet penetrated by channels filled with low-compressibility fluid. Metrological tests have shown that it is several times more selective for normal stresses than are rigid disc-type detectors.

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EXPERIENCE OF MEASURING REACTIVE REPULSE IN THE FOUNDATION OF BUILDINGS WITH THE HELP OF STRINGED DYNAMOMETERS. Kleshchev P.E., Muller R.A.(USSR).

In order to measure strains, arising from the contact of structures with the ground bedding, meters of various construction founded on the stringed method of measuring deformations and strains, being simple in construction and proving stability of readings in tine are used on a much larger scale at the present time. In particular, to such instruments, as experience of their usage in investigating the nature of character of redistribution of reactive repulse of the ground along the foot of the belt foundations of buildings in the Karaganda coal basin has shown, stringed dynamometers of DS-10 type, constructed by VNIMI with hoisting capacity of 10 tons may be referred. Dynamometers in the process of construction of buildings were mounted along the foot of the foundations upon supporting armoured concrete slabs(pads) according to the diagram on fig.1. In the result of which the building rested completely upon dynamometric blocks(pads) and had no free contact with the bedding.

This allowed to obtain a continuous picture of the epure of the ground repulse along the foot of the foundations up to a complete stabilization of uneven settlings of the building(fig.2)[x] to determine the quantity of generalized moments of deflection and torsional moments and transversal forces actually acting upon the building and to investigate the interaction between the building and the deforming foundation.

The index of trustworthiness of data obtained from the readings, of the instruments arranged according to the diagram, is quite a close(fig.3) coincidence of actual weight of the building, determined in the process of its construction by weighing all its constructions, with summary measured epure of repulse of the ground, obtained by summarizing the readings of all the dynamometers at the moment of each observation.

As it is easy to be convinced by examining fig.3, discrepancy between the actual and the measured on the dynamometers, indications of the weight of the building, from the moment of completing its construction, amount on the average to 5%. The investigations carried out confirm the suitability of using stringed dynamometers for carrying out prolonged investigations on the interaction of a building with a deforming foundation and give grounds to be credible to the data obtained with their help.

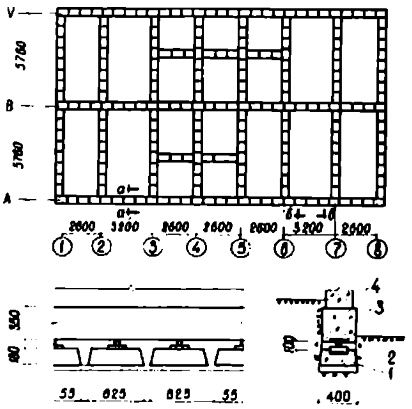

Fig.I Sceme of the arrangement of dynamometers along the foot of the foundations. (I - supporting reinforced concrete slab, 2-dynamometer DS-IO, 3-reinforced concrete foundation belt, 4-ground floor panel)

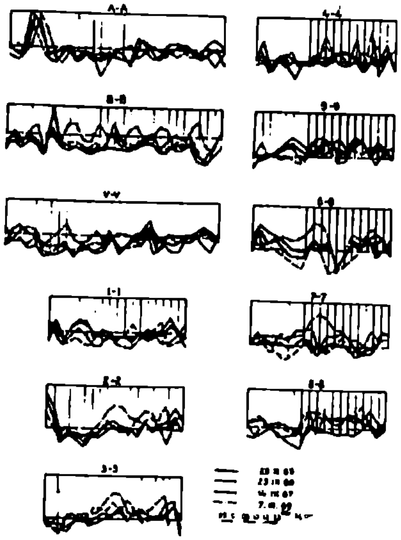

Fig.2 Diagram of reactive soil's repulse on typical dates of observations.

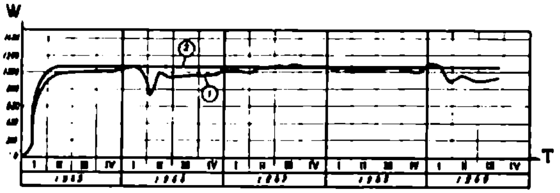

Fig.3 Comparison of summary measured diagram of soil's repulse along the foot of foundations (curve I) with the weight of house No.2 (curve 2).

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[x] On fig.2 the epures of actual repulse of the ground under all the main walls of a building on individual characteristic dates of observation are shown.

AN EXAMPLE OF USE OF INCLINOMETER IN CONNECTION WITH LATERAL LOAD TESTS ON PILES.

M.B. Jamiolkowski (Italy) and S. Marchetti (Italy)

Two full lateral load tests on open end driven steel tubular piles penetrating into the soft N.C. marine clay were performed in Manfredonia (Southern Italy).

The characteristics of one of these piles and soil properties are:

- Pile geometry: O.D. = 122 cm; Wall thickness = 2,22 cm, Penetration = 22 m, Distance load-mud line = 15,4 m.
- Soil properties; Undrained shear strength, c_u = 0,02 + 0,00022 z kg/cm^2; Compressibility modulus E_{ed} = 8,5 $\bar{\sigma}_{vo}$; Atterberg Limits, L.L. = 75, P.I. = 55.
  being: z = depth below mud line in cms; $\bar{\sigma}_{vo}$ = effective overburden stress.

Each of these two piles were instrumented with inclinometer tubes in order to measure the pile axis rotation during each step of loading, using high precision Digitilt Inclinometer - Model 50301.

Interpretation of inclinometer readings was done by means of regression analysis using polynomials of degree varying between 5 and 8.

One integration and three derivations of the interpolated polynomial functions to obtain for each step of loading pile deflection (y), bending moment (M), shear (T), and soil reaction (P) values with depth were used.

One example(1) of such calculations is shown in figure 1.

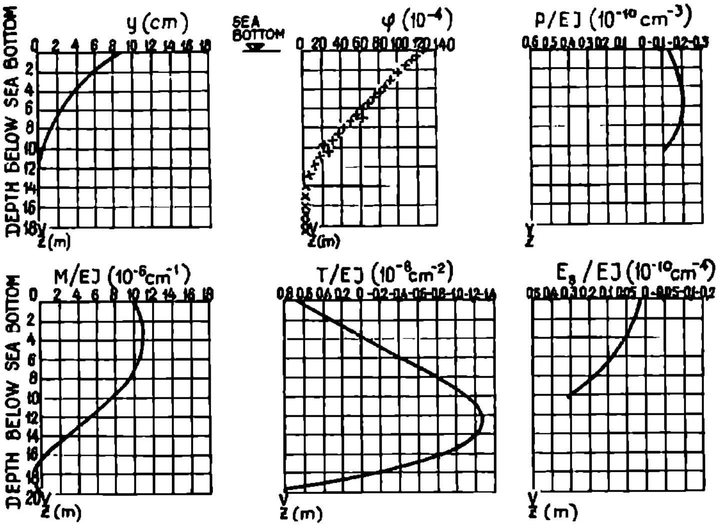

FIG. 1 - MANFREDONIA PILE D_e = 122 cms
PILE DEFLECTION (y), BENDING MOMENT (M), SHEAR (T) AND SOIL REACTION (P) AS OBTAINED ON THE BASIS OF MEASURED PILE AXIS ROTATIONS (φ) FOR H = 20 TONS

In figures 2 is given the soil modulus $E_s = \frac{p}{y}$ (FL^{-2}) versus depth, as evaluated from inclinometer readings. In the same figure, values of initial tangent soil Jlus E_{st}(2) as extrapolated from P/y curves are o shown.

The following considerations may be drawn:

1. The use of inclinometers can give results useful in the study of interaction between soft soils and laterally loaded piles in problems connected with tubes installation(3), and equipment calibrations are carefully examined.
2. Further work should be done regarding the validity of different interpretation methods to be used in the interpolation of inclinometers readings in the study of soil-structure interaction.
3. Values of soil moduli obtained from interpretation of inclinometers readings are in agreement with general soil properties previously given and with results of similar studies reported in the existing literature.
4. For all the examined cases the soil modulus increases more than proportionally with depth.
5. The load dependent shape of E_s = f (z) as shown in figures 2 and 4 clearly indicates non linear soil behaviour even for small loads.

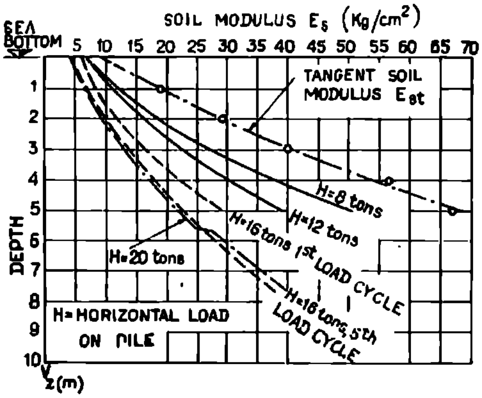

FIG 2 - MANFREDONIA PILE D_e=122 cms
VARIATION OF SOIL MODULUS WITH DEPTH

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(1) Full results of these tests are reported in the paper submitted by the Authors to XI Italian Conf. on S.M. and F.E. held in Milan (Italy) 1 ÷ 4 March 1973.

(2) E_{st} is the E_s values for y = 0; the extrapolation was obtained using normalized hyperbolic P/y curves representation.

(3) With particular reference to minimization and evaluation of spiral effects.

STRAIN METER FOR MEASUREMENT OF VERTICAL DEFORMATION BENEATH EXCAVATIONS AND STRUCTURES. Bill Jewsbury (U.S.A.)

GENERAL: It is not always possible to erect modern structures on completely ideal foundation materials. In most cases, excavation is necessary to remove unsuitable materials. This is followed by construction of the foundation elements and then the erection of the structure. Accurate measurements of heave during excavation and subsequent settlement during construction are very important to the engineer to validate design assumptions.

An instrument has been developed for this purpose and has been utilized on several projects over the past few years. This service has demonstrated achievement of the intended design objectives. These are: 1) Remote reading electrical sensing and readout; 2) Relatively inexpensive; 3) Simplicity; 4) Reliability; 5) Ease of installation.

DESCRIPTION: The instrument described in this paper is shown in Figure 1. It utilizes an electrical sensing element which is a linear potentiometer. This variable resistance element is conductive plastic material and not subject to the limitations of wire-wound elements. The movable contact is mounted on a 1/4-inch stainless steel shaft which slides within the housing sealed by an O-ring.

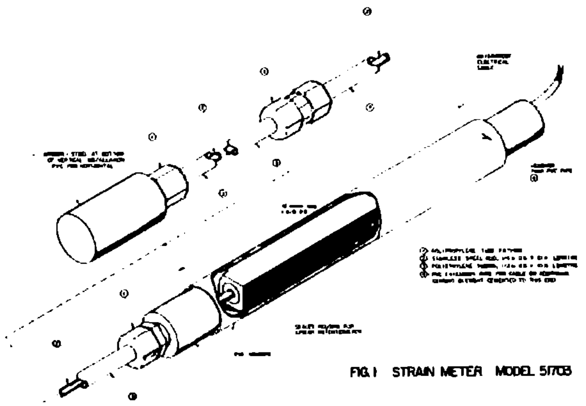


FIG. 1 STRAIN METER MODEL 51703

Standard displacement range of the instrument is 6 inches. Linearity is ±1% of the range. Other ranges are at the user's option.

The potentiometer and its electrical connections are potted and sealed within a protective enclosure of PolyVinylChloride pipe. A rod 1/4-inch in diameter connects the potentiometer shaft to the anchor point. The rod material is normally stainless steel. To eliminate temperature effects on long lengths, Invar rod is used. The rod is enclosed within polyethylene tubing.

When the 51703 strain meter is used for measuring heave under excavations, it is coupled to PVC riser pipe to the surface. Coiled electrical leads are enclosed in this riser pipe and can be extended more than six times the closed length. This allows the leads to be lowered into the pipe when not in use and they can be retrieved for reading. In case the riser pipes are damaged by excavation equipment, the leads are recoverable.

The readout, Figure 2, used with the strain meter is a portable, battery-powered unit with a digital indicator reading from 1 to 1000 dial units. It utilizes a Wheatstone-bridge circuit which is manually balanced until a null condition is achieved as indicated on the panel mounted galvanometer. At this point, the reading of the dial indicator is recorded. The dial reading for 6-inch range is .006-in. per dial unit. The strain meters are connected to the portable indicator by multi-conductor cables either directly or through a terminal box.

FIG. 2 STRAIN INDICATOR

INSTALLATION: The 51703 vertical strain meters are normally installed in drilled holes prior to excavation. The extension rods and tubing enclosure are assembled as the device is lowered into the hole. The units are grouted in place with a weak cement/lime grout.

The minimum size hole or casing for a single unit is 3-inch diameter. A typical installation is shown in Figure 3. Many variations are possible.

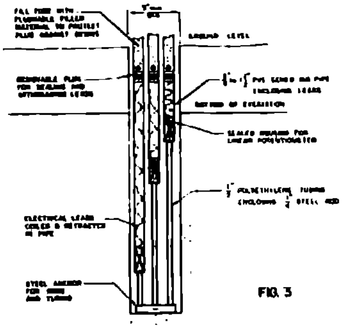
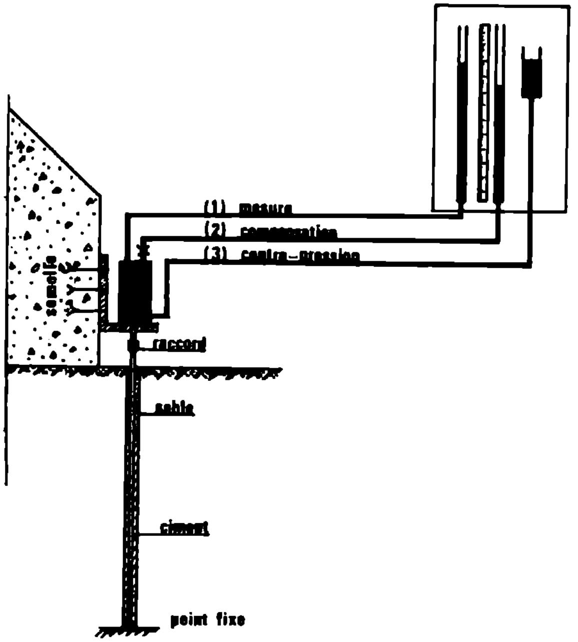


FIG. 3

AMPLIFICATEUR DE TASSEMENT A VERIN POUR LA MESURE DES TASSEMENTS SUR OUVRAGES.

H. Lemasson et JC. Carlo - (France)

Dans le but de mesurer avec une bonne précision les tassements des ouvrages d'art, qui sont généralement de faible amplitude, nous avons conçu et expérimenté un nouvel appareillage qui est décrit sur la figure suivante.



Un corps de verin, de section S, est fixé à la structure que l'on veut suivre, généralement au niveau de la semelle. La tige du piston de ce vérin est reliée par une transmission rigide à un point fixe, que l'on réalise par forage puis scellement d'une tige sur un horizon incompressible. Le corps du vérin est relié par un tuyau souple (1) à un tube de lecture de section s en surface.

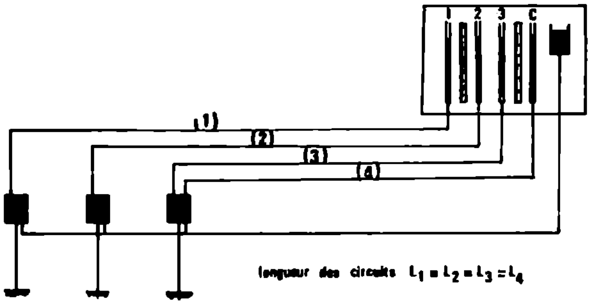
Un tassement de la structure se traduit par une expulsion d'un certain volume d'huile du corps de vérin et se manifeste en surface par une élévation du niveau dans le tube de lecture. Le rapport d'amplification du tassement est donné par le rapport des sections $\frac{S}{s}$.

Les perturbations causées par les phénomènes atmosphériques, tels que dilatation ou contraction des tuyaux souples de jonction, ou évaporation du liquide antigel au niveau du tube de lecture, sont corrigées par une seconde lecture, faite sur un circuit (2) identique en tout point au premier, mais ne communiquant pas avec le corps du vérin, dit circuit de compensation.

Par ailleurs, le système doit être parfaitement étanche, aussi pour éviter les fuites à long terme sous les joints du piston, prend-on la précaution de faire travailler le vérin à pression nulle en appliquant une contre-pression par le circuit (3).
Le liquide de remplissage de l'ensemble des circuits est du liquide antigel plutôt que de l'huile, car les purges sont plus faciles à réaliser.
L'intérêt de la méthode réside dans les points suivants :

- la précision peut être comparée à celle des meilleurs niveaux optiques actuellement connus. Les limitations de ces méthodes optiques, comme la diffraction par forte chaleur où les vibrations transmises aux appareils par le vent ne gênent pas ce type de mesure.
- la lecture peut être suivie par un personnel non spécialisé.
- le prix de revient d'un tel appareillage est relativement modeste.
- le principe de lecture directe peut permettre de suivre certains phénomènes en dynamique.

Il est possible, si plusieurs points de mesures sont à prendre sur un même chantier, de concevoir des panneaux de lecture pouvant alimenter plusieurs amplificateurs avec une seule contre-pression et une seule mesure de compensation, comme le montre le schéma ci-dessous à 3 amplificateurs.



Il faut néanmoins que tous les circuits de mesure et de compensation aient le même cheminement, dans la partie hors de la terre surtout, si l'on veut que la mesure de compensation ait un sens.
Le dimensionnement du vérin, longueur de course et section, est fonction de l'amplitude du tassement à mesurer et de la précision que l'on désire obtenir.

L'expérimentation de ce type d'appareil de mesure a été effectuée sur un ouvrage important à MORLAIX. La course du vérin était de 5 cm et le rapport d'amplification de 20. La précision de la mesure a été en pratique de l'ordre du 1/10ème de millimètre pour des tassements de 2 cm échelonnés sur deux années.

REMOTE READING INSTRUMENTATION FOR LARGE PLATE BEARING TESTS.

R. MILLER AND F. BROWN (USA)

Large plate bearing tests were conducted against the vertical side faces of an elongated test pit at a depth of 58 feet below street level, to determine the deformation characteristics of a poorly cemented sandstone, and to verify the acceptability of a bearing pressure of 25 tons per square foot recommended for the design of a large tower footing for a planned multi-storied building. To obtain the greatest amount of information from the tests, deflections were monitored at 18 locations simultaneously, or 9 positions each on opposite test faces. This paper describes the test equipment and the unique remote reading instrumentation used for monitoring rock deformations under this restricted urban site.

The bearing tests incorporated two 34-inch diameter Freyssi flat jack flexible pads positioned on opposite rock faces and reacting against a rigid system of bearing plates and four 10-inch diameter adjustable pipe struts. The details of the test set-up are shown in Fig. 1. Fine adjustments of extra space between the bearing plates and the pipe struts were accomplished by four screw jacks. Load was applied to the rock face by pumping hydraulic oil into the two flexible flat jacks.

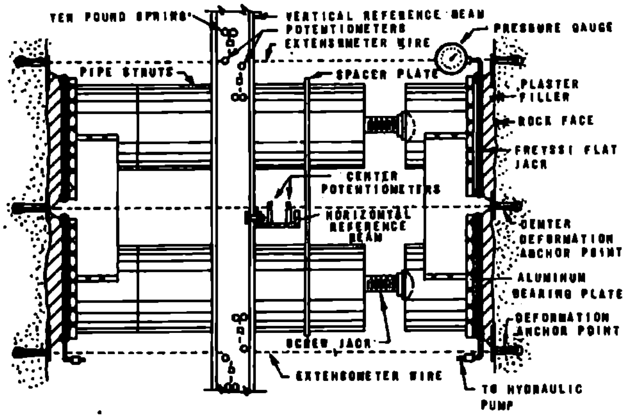


Fig. 1 Section Through Jack Test Set-Up

Deformation of the sandstone under applied pressures was measured by electrical extensometers, equipped for remote read-out at the top of the test pit. Five of these extensometers were placed to measure deformation within the loaded area on each face, one at the center point through a 2-inch diameter hole in each flat jack, and four at the quarter points on the perimeter of each jack. Four other extensometers were also placed at the quarter points around each jack, but at one diameter from the edge of the loaded area. The locations of five of the recording points are shown in the cross section in Fig. 2. The remaining four points are positioned the same distances apart but perpendicular to point 3.

At the nine recording points on each test face, extensometer deformation anchors for measuring deflection were set approximately 3 inches into each rock face. These anchors, shown in Fig. 1, consisted of 6-inch long, 1/4-inch diameter expansion bolts, which were placed and expanded in 1/2-inch diameter holes predrilled in the sandstone. For those points in the center or on the edges of the bearing pad, metal plates were also placed over the expansion bolts and extended under the loaded area. The upper end of the bolts were threaded so that the plates could be tightened firmly to the face of the rock with a nut. These plates provided added assurance that the anchor point would remain fixed in the rock wall even if the anchor expansion bolt were to slip when large stresses were applied to the surrounding rock. In the head of each expansion bolt, a small hole was drilled and a screw-tapped head was mounted to fix the extensometer wire firmly to each anchor bolt.

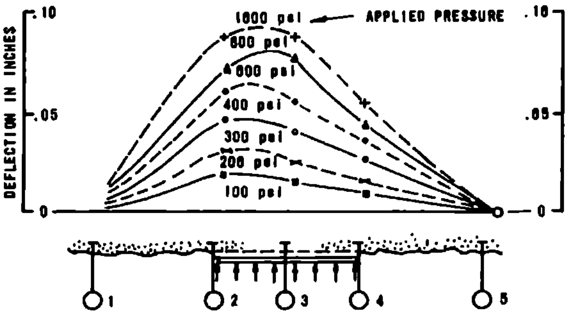


Fig. 2 Cross Section And Deflection Of Rock Face

The electrical portion of each extensometer assembly consisted of a potentiometer mounted to either vertical or horizontally-oriented 5-inch steel channel beams, which served as a reference base. These channels were bolted together in the center between, and free from the jack-reaction struts, and on the ends at points at least 7.5 feet from the test centerline. Each extensometer was connected to one deformation anchor point with a fine, flexible, stainless steel wire about 2 feet long, passing perpendicular to the test face around a wheel on the potentiometer. Tension was maintained in each extensometer assembly by attaching the other end of the wire to a constant-tension negator spring also mounted on the reference base. Each spring had a calibrated 10-pound pull. The change in distance between the reference base extensometer assembly and each anchor point was measured by balancing a Wheatstone bridge circuit, the reading being indicated on a digital readout potentiometer contained in a portable unit at the ground surface. Each dial unit on the readout was equivalent to two thousandths inch (0.002") of face deflection. The maximum range of each potentiometer was 1000 units, or 2 inches of total measurable deformation for each anchor point, without resetting of the potentiometers.

All electrical leads attached to the potentiometers were extended to the surface above the test pit where all deformation readings were taken remotely. Working from the surface provided extra safety to the recorder from possible caving during loading, enabled readings to be taken rapidly and in a consistent sequence, and eliminated the risk of disturbing or bumping the test equipment as the tests were conducted. In addition to remote recording, this type of deformation instrumentation also has the advantage of enabling deflection measurements in the loaded area to be made directly at the rock surface. For this flexible loading system, direct measurements eliminate corrections for deformation of the bearing plates and plaster leveling pads.

A typical plot of deflections obtained at various loading pressures is presented in Fig. 2. Except for minor variations the pattern of deformations of a flexible plate appear reasonable, further indicating that this method proved to be an excellent application for measuring deflections for these large plate bearing tests.

AN INSTALLATION FOR DETERMINING STRESSES AND DEFORMATIONS OF A LAYER OF CLAYEY SOIL ON A RIGID BASE.

A.L.Mindich,S.S.Vyalov (USSR)

The installation was devised for investigating the stressed-strained state developed in a layer of clayey soil of finite thickness when loaded by a rigid strip-type test plate. The set of instruments employed in the installation provide for the investigation of the following:the contact stresses at the bottom surface of the plate and at the surface of the rigid base,the stress at various points within the layer of soil,the displacement of points within the layer and on its surface and the displacement of the test plate.

The installation consists of a tray,test plates,stress gauges and a device for determining the displacement of particles of soil in the layer.

The tray is 1940x820x600mm in size. The width of the test plates B=150,225 and 300mm The length of the plates corresponded to the tray width.The metal plate serving as the bottom of the tray was used as the rigid base.

Pressure gauges were mounted in the bottoms of the tray and test plate,flush with the surface.They were used to observe the variation in contact stresses during the tests.At certain points of plane XZ, normal to the axis of the test plate (axis Y) gauges were embedded for measuring the stresses $\sigma_x, \sigma_y, \sigma_y$, and σ_ψ (in a direction inclined at equal angles to axes X and Z).For layers 75 and 150 mm thick,the gauges were arranged in the middle of the layer along the axis of summetry of the test plate and at distances of 150,300 and 450mm from this axis.For layers 300 and 450mm thick,the gauges were arranged, respectively, at two and three levels.

Tests were conducted on koalin (unit weight 1,86 g/cm^3,water content 36%,consistency index 0.41, initial modulus of total deformation 25 kg/cm^2, angle of internal friction 18°, cohesion 0.07 kg/cm^2).The soil layers were H=75,150,300 and 450 mm thick.

Stress Measurements

The instrument,whose design was developed together with Yu.K.Tkachev, consists of two identical disks, 33mm in diameter and 1.1mm thick,having an annular bead on their perimeter.The two disks are joined on their perimeters by soldering. This enables both sides of the instrument to participate to an equal extent in determining the parameter being measured, owing to which the effect of orientation is practically excluded. The deformation of the disks is transmitted to the measuring member, an elastic blade 0.1mm thick on both sides of which sensing elements are glued. The span of the measuring member is 2,5 mm Consequently,its curvature upon being deflected is equal to the sum of the deflections of the two disks and considerably exceeds the curvature of the latter. As a result the gauge has a sensitivity of 0.01 to 0.02 kg/cm^2. The deflection of the centre of the disk is 2.7x10^{-4}cm for a load of 1 kg/cm^2.

Determining the Displacement of Particles of the Soil Layer

This device,developed together with L.B.Glezin,consists of two thin flat screens arranged in the plane of motion of the soil particles. During the tests one screen is stationary and the other moves together with the test plate.This enables the motion of the soil particles to be investigaed simultaneously in two systems of coordinates. The two screens are located in an elastic latex cover envelope which has needles inside and rigid elements, moving together with the soil, on the outside. Upon variations in the pressure inside the envelope,the needles approach or are retracted from the screens, leaving traces on them. The pressure is changed by connecting a vacuum pump to the envelopes at given instants of time. This enables the mean velocity vector of particle displacement to be determined.

The Results

The application of the above-described instruments enabled a picture of the development of the stressed-strained state to be obtained for a layer of clayey soil on a rigid base.

1. The character of the development of the contact stresses and the component of the stress tensor depends both on the reduced layer thickness (λ =H/B) and on the degree with which the load on the test plate (P_m)approaches the limiting value (P_{lim})at which the soil loses its bearing capacity. With an increase in load the deformation of the soil proceeds in several stages,from an "elastic" stage to a "plastic"one: the interval of loads corresponding to the "elastic" stage depend on λ . For comparatively thin layers of soil (λ<1),the diagram of contact stresses under the bottom surface of the plate and the stresses inside the layer correspond to the elastic solution at loads up to P_m(0.1 to 0 0.15) P_{lim}. As the load increases to P_m(0.5 to 0.6)P_{lim},the shape of the contact stress diagram changes. It becomes "parabolic"with a maximum along the test plate axis.As the load approaches the limiting value,the diagram approaches a shape corresponding to the theory of plasticity. The stresses within the layer are characterized by a considerable increase in the horizontal components. This leads to a substantial turning of the axes of the principal stresses (up to 70°).

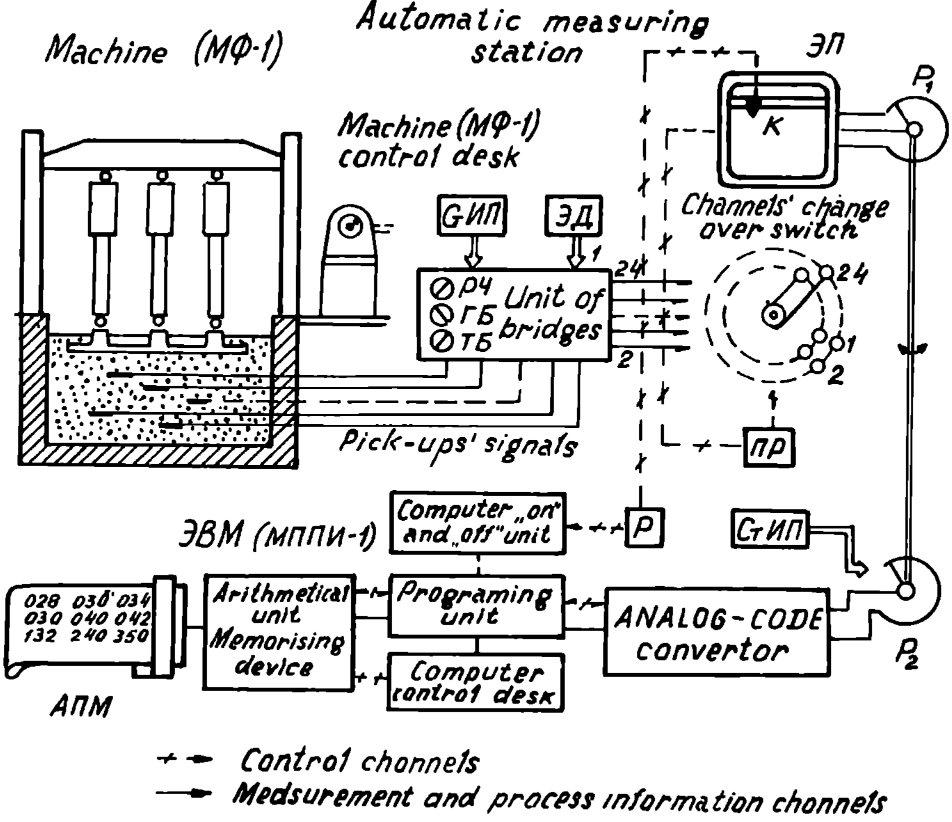
The experiments showed that the distribution of the contact stresses over the surface of the rigid base approximates the well-known theoretical solution.

2. The pictures of the displacement of points within the layer and on its surface indicated considerable development of displacement in a horizontal direction from the axis of the test plate. The thinner the layer,the greater the displacement. The configuration of the stationary zones under the bottom surface of the test plate and on the surface of the rigid base was revealed, as well as the boundaries of the protrusion zone.

AUTOMATICALLY CONTROLLED TESTING COMPLEX USING A DIGITAL COMPUTER DESIGNED FOR ON-THE-MODEL EXPERIMENTAL INVESTIGATIONS OF GROUNDS AND FOUNDATIONS. Y.N.Murzenko, V.V.Revenko (USSR)

In the laboratory of grounds and foundations of the Novocherkassk Polytechnical Institute there is accomplished an automatically controlled testing complex using a digital computer (ЭВМ) intended for carrying out experimental investigations of grounds and foundations upon the large-scale models.

The testing complex includes the following units (see appendix): a testing machine (МФ-I) instruments and pick-ups;automatic measuring unit and a specialized digital computer.



The testing machine is designed for carrying out experimental on-the-model investigations of grounds and foundations.

Three-dimentional chute of the machine has the following dimensions:5x3x2,2(height). It is filled with ground.The power frame of the machine is equipped with three specially designed hydraulic jacks intended for loading models of foundations.The remote control panel includes as follows: a hydraulic system pumping unit;dynamometer and the elements of automation providing normal rating of the machine and stabilizing loading with time.

Devices and pick-ups are developed for measuring and the remote-controlled registration of a model loading,general shifts and deflections,stresses inside the model,stresses at the contact and in the mass of the ground as well as in other parameters.

The automatic measuring unit is accomplished as a stationary and multichannel one and is provided with a highly stabilized power source (СтИП),units of tensometric bridges with the sensitivity governors (РЧ)of rough and precise balancing (ГТ И ТБ),standard pick-up (ЭД)and the automatic recorders (ЭП) with the control desk assembly (П Р),By means of the automatic recorders the results of measurements are registered and the analog signals to be introduced into the digital computer are formed simultaneously.A governing digital computer permitting processing results of measurements immediately in the course of an experiment is installed in the testing complex intended to solve the problem of automatic performance of all the process.A number of problems are settled to enable a digital computer to be applied in the testing complex.They are as follows: a)the principle of forming necessary level of the analog signal: tensounit-computer (by means of rheostats with slide P_1P_2);b) program-circuit conjugation of the computer with the measuring unit (by means of the contact K,relay P the computer "on" and "off" unit and the corresponding control program);c) choice of throughout the complex rating and the operation program projecting.

Here are the two parts of the digital computer operation program:the pick-ups calibrating program and the program of an experiment.

At every stage of calibrating instruments and pick-ups to be used in the experiment the computer performs the program inquiry of signals and memorizes them.

On finishing calibrating the calibration characteristics are calculated for each pick-up by the method of the least square,the latter being kept in the operation memory and simultaneously produced for typing.

In the process of the experiment the computer keeps inquiring the pick-ups' signals,evaluates their increment,decodes the readings from the calibration characteristics of each pick-up and using an automatic type writer (АПМ) self-actingly produces the obtained results of the measurements per stage of loading in the shape of a systematized table of wide digital writing.

In the progress of accomplishing the experiments there is ascertained high efficiency of operation of the automatically controlled testing complex as well as the possibility of sophisticated tests accomplishment alongside with the synchronous measurement of various parameters of a foundation model and the ground working jointly.

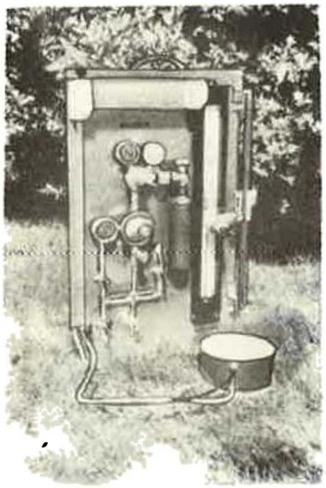
Analyses of the time expenses indicated that the labourconsuming of the primary processing of the measurement results with the digital computer used reduces 270 times roughly as compared with the conventional methods of processing.

MESURE EN PLACE DES TASSEMENTS SOUS LES REMBLAIS ET OUVRAGES D'ART - LE TASSOMETRE M. Peignaud (France)

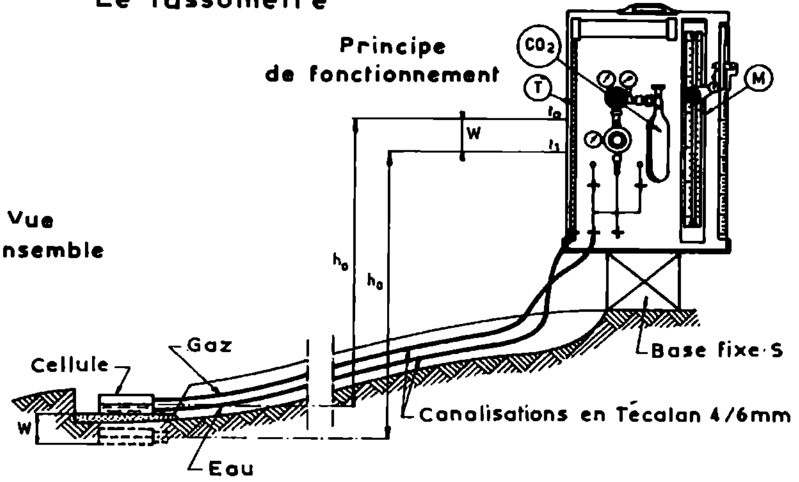
L'appareil mis au point au Laboratoire Régional des Ponts et Chaussées d'Angers (France) permet de mesurer le tassement du sol pendant et après la construction des ouvrages.

Une cellule de prise de niveau, placée dans le sol, sous l'ouvrage, suit tous les mouvements imposés à celui-ci. La position en altitude de cette cellule, par rapport à un repère fixe S, hors de la zone soumise au tassement, est suivie à l'aide d'un panneau de mesure reposant sur ce repère fixe S.

Le Tassomètre



Vue d'ensemble



Principe de fonctionnement

Principe de la mesure

La cellule étant à demi remplie d'eau, on applique sur le liquide une pression p_0 (en général du gaz carbonique) de façon à refouler le liquide dans le haut du tube de lecture T où la lecture, après stabilisation du niveau du liquide, sera t_0 sur une échelle graduée en mm. Lors de la première mesure, après chargement du sol (le panneau reposant à la même place sur la borne-repère S) on applique dans la cellule, la même pression p_0 et la lecture dans le tube T sera t_1.

Le déplacement de la cellule, donc le tassement, sera: $W = t_0 - t_1$ (en mm).

Quand le tassement est trop important, (la lecture t étant en bas du tube T) on change la pression appliquée dans la cellule, $p_1 > p_0$.

Appareillage

Les cellules, perdues dans le sol, sont en chlorure de polyvinyle. C'est un cylindre de 160 mm de diamètre et de 90 mm de hauteur. Elles sont équipées de raccords à serrage rapide pour les canalisations de gaz et de liquide. Il existe un modèle spécial "cellule de profondeur" qui peut être placé dans un forage pour la mesure du tassement à différentes profondeurs aux interfaces des différentes couches.

Le panneau de mesure, facilement transportable d'un point fixe à un autre, pèse environ 15 daN. Il comporte les organes suivants :

- mise en pression : réserve de gaz - détendeurs - régulateur ;
- dispositif de mesure de la pression : manomètre à mercure avec lunette de visée sur vernier au 1/50e de mm ;
- tube de lecture du tassement : (échelle graduée en mm) ;
- la liaison entre la cellule et le panneau est assurée par deux tubes souples en matière plastique de 4 mm de diamètre intérieur munis de raccords rapides.

Mise en oeuvre - Performances

Les mesures peuvent être faites avec des distances panneau - cellule dépassant 100 m. Le nouveau modèle permet de mesurer des tassements atteignant 6 m avec une précision absolue de 1 cm. En prenant des précautions spéciales, pour les ouvrages d'art, la précision absolue peut être de 2 mm. Les cellules et canalisations de raccordement sont enterrées à l'avance, donc ne gênent pas les terrassements et les mesures peuvent être poursuivies après mise en service des ouvrages.

Le panneau de mesure, livré dans une valise de transport et de protection, est utilisé pour plusieurs cellules, sur différents chantiers. Il est raccordé juste au moment d'effectuer les mesures. Le premier appareil, fonctionnant depuis 1965, a permis de mesurer des tassements de 2,20 m d'amplitude, sous un remblai routier.

LA MESURE DES CONTRAINTES SOUS REMBLAIS ET OUVRAGES D'ART

M. Peignaud (France)

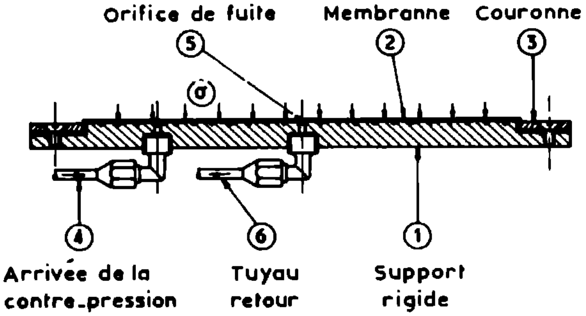
CAPTEUR DE PRESSION TOTALE DE SOL A CONTREPRESSION PNEUMATIQUE ET DEBIT CONTROLE

Les mesures de contraintes dans les sols et, notamment, des pressions appliquées par les ouvrages sur le sol de fondation, durant plusieurs mois ou même plusieurs années, exigent l'emploi d'appareils fidèles ne présentant pas de variations de leurs caractéristiques dans le temps (dérive). Pour faire de nombreuses mesures ces capteurs doivent être bon marché, aussi le Laboratoire d'Angers a-t-il cherché une solution évitant l'emploi de l'électronique.

Principe de fonctionnement du capteur de pression totale

L'ensemble de mesure comporte une cellule de prise de pression (capteur) et un panneau de mesure transportable. La cellule est placée sous un ouvrage ou contre une paroi (butée - poussée). Elle comprend un support rigide et une membrane très mince, fixée sur le support. La cellule se présente sous la forme d'un disque, très plat, de 215 mm de diamètre et de 7 mm d'épaisseur, en matière inoxydable. Au repos, la membrane obture l'orifice d'arrivée (4). Une contrepression de gaz est envoyée par cet orifice, le gaz se répartit uniformément sous la membrane et lorsque la pression appliquée dans le capteur équilibre la pression du sol, le gaz s'échappe par l'orifice de fuite (5) et retourne au débitmètre du panneau de mesure. La pression appliquée se stabilise à une valeur p, lue sur le manomètre du panneau, égale à la pression des terres σ.

Capteur de pression totale de sol à contre-pression pneumatique et débit contrôlé



Coupe de la cellule

Modèles de 215 mm et 50 mm de diamètres

Appareillage

Deux modèles ont été réalisés ayant 215 mm et 50 mm de diamètre (épaisseur 6 mm et 3 mm), ils sont réalisés en acier inoxydable. La liaison cellule - panneau de mesure est assurée par deux canalisations en matière plastique de diamètre 4 mm (intérieur).

Le panneau de mesure comprend les organes suivants :
- mise en pression : réserve de gaz - détendeurs - régulateurs ;
- débitmètre de précision pour contrôle du débit de fuite ;
- manomètre de lecture de pression.

L'ensemble est réuni dans une valise de transport.

Mise en oeuvre - Performances

Le capteur est enterré sous l'ouvrage ainsi que les canalisations de raccordement. Pour effectuer une mesure, on raccorde la cellule au panneau (raccords rapides) et, après réglage du régulateur, un peu au-dessus de la pression à mesurer, on ouvre une vanne aiguille. La pression lue au manomètre croît, puis se stabilise à la pression des terres s'exerçant sur la cellule. D'une mesure à l'autre, le débit de fuite est contrôlé par un débitmètre.

Le capteur peut mesurer des pressions jusqu'à 5 bars, la précision est celle du manomètre, c'est-à-dire de l'ordre de 0,5 % (de l'étendue de mesure). Il peut marcher en continu, avec une légère fuite permanente, on peut alors adjoindre un enregistreur. En fonctionnement discontinu, la flèche de la membrane, sous pression, est de 4.10^{-2} mm, soit un rapport diamètre-flèche de l'ordre de 5 000. En continu, avec fuite permanente, ce rapport dépasse 10 000. L'appareil est très fidèle car la membrane se déforme très peu et demeure soutenue, au repos, par le support rigide.

Des essais sont actuellement en cours pour tester la réponse des capteurs dans les milieux granulaires à l'aide d'une enceinte de 0,8 m de diamètre et de 1 m de hauteur. Le matériau est contenu par une enveloppe de caoutchouc et il est possible d'appliquer un champ de contraintes connu et de mesurer les déformations latérales.

A HIGH SENSITIVITY PIEZOMETER

C. Silva E. (México)

INTRODUCTION

The apparatus described herein is a membrane piezometer operated by a compressed gas. The design is based upon ideas developed by Marsal and Ramírez de Arellano (1965).

The reliability of this instrument with a total volume displacement of 0.3 cu cm was determined by means of laboratory test. Its behavior has proved satisfactory and it has been installed at La Angostura dam, Chiapas, México (Silva, 1973).

DESCRIPTION

The main body of the instrument is composed by two cylinders, parts A and B (Fig 1), made of stainless steel, or polyvinyl chloride (P.V.C.) joined by six screws; the operating gas lines arrive to cylinder A, while the porous stone and the protective filter are attached to cylinder B. The thin diaphragm is sealed by means of O-rings between parts A and B, and piece C, is fixed to the diaphragm (Fig 2). The latter is die-stamped and made of 0.002 in. --thick stainless steel shim or 0.007 in.–thick teflon sheet, type TFP. The shape of the diaphragm notably reduces edge-effects, and the close fit of the seal components allows the use of very thin membranes, even for rather large differential pressures.

OPERATION

Measurements are made by injecting an inert, dry compressed gas into the intake line until the diaphragm flexes, thus allowing the passage of air through the outlet line, which is connected to a set of pressure gages. The gas supply is then shut off. Gas is then bled through a valve connected to the intake line. Whereupon pressure drops at a rate of approximately 10 gr/sq cm/sec. When the diaphragm subjected to the pore pressure returns to its original position, the outlet line is sealed off and the pressure reading is taken.

TESTS

The prototype was subjected to both weathering and structural strength tests. Following each test, the apparatus was calibrated. The results obtained were satisfactory in all cases, since calibration changes were insignificant (less than 0.002 kg/sq cm).

The weathering tests consisted of:

a) Submerging the diaphragm in pore water extracted from Texcoco Lake clays (pH = 10) for 30 days.

b) Subjecting the apparatus to 5 000 cycles of wetting in the above water and drying at a temperature of 40 °C.

The structural strength tests were as follows:

a) A pressure of 40 kg/sq cm was applied for 30 days on the porous stone side, while the other side was connected to the atmosphere.

b) Under the above pressure, the diaphragm was subjected to 2 000 loading-unloading cycles, each representing the process of taking a reading.

c) A pressure difference of 5.0 kg/sq cm was applied between the inlet and the porous stone side representing an accidental rise, not likely to occur during normal operation.

CALIBRATION AND SENSITIVITY

Measurements below 6 kg/sq cm were made with a mercury manometer; for higher pressures an 8 1/2 in. Helicoid manometer was used.

The minimum detectable pressure is of 0.01 kg/sq cm.

The maximum difference between various readings for the same pressure was ± 0.003 kg/sq cm.

The above results correspond to the testing of ten different diaphragms of both stainless steel and teflon.

The pressure ranges for which the instrument was designed are:

0 to 40 kg/sq cm (Stainless steel diaphragms)
0 to 10 kg/sq cm (Teflon diaphragms)

ACKNOWLEDGEMENT

The author gratefully acknowledges the suggestions of F. Santoyo and L. Ramírez de Arellano during the development of the instrument herein described.

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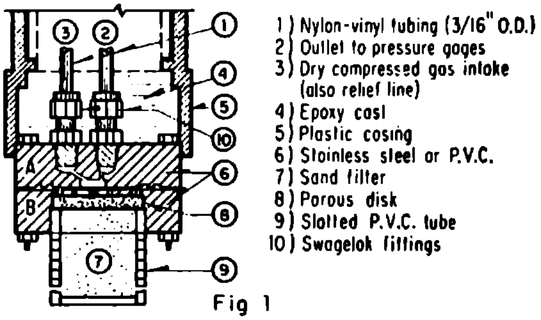

Fig 1

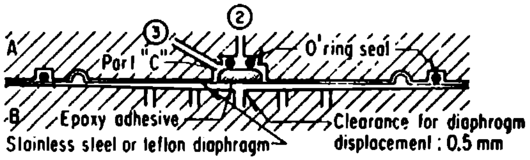

Fig 2

NUCLEAR AND RADIOISOTOPE TECHNIQUES FOR DETERMINATION OF SOIL PROPERTIES IN CONSTRUCTION

M.I.Smorodinov, I.V.Lavrov, V.E.Romanchuk, M.N.Okulova, G.S.Goskova, (USSR)

Nuclear and radioisotope methods have a series advantages comparatively with the applied now methods of determination of soil properties: the possibility of soil investigation without distortion of their structure and in a greater volume than for the well-known methods; fastness and reliability of obtaining of the information, the possibility of repeatable and regime observations.

Investigation and application of the radioisotope methods in the construction in USSR promotes in some lines:

1. Designing of the radioisotope devices for determination of density and moisture con tent in bore holes and on the surface of soil, including the designing of specialized gauges for soils with obvious specific features. All the types of gauges must combine the high sensitivity to measuring parameters with small output of radioactive cources. From number of depth gauges for measurement in bore holes the assemble of radioisotope gauges, consisting of PB -36 (neutron moisture meter), PB -36 (gamma-density meter) and counting device ought to be mentioned. Diameter of the depth probes-36mm, the length of cabel-20m. the gamma-ray source- is of 0.25 mg/eqv.Ra., the plutonium-berillium neutron source is of 1.10^5 n/ces. From number of gauges for measurements on soil surface or at the depth till 30-50cm, we can call the combined radioisotope density-moisture meter PB -1 (for homogeneous soils) and gamma transmission density meter (for determination of density of coarse-rubbled soils and other soils of heterogeneous structure).

Under the conditions of field measurements adhering to calibration ones, the guaranteed accuracy of soil density determination is about $\pm 0,05$ g/cm^3 and accuracy of moisture content determination is about 0,5% in the range 1-10% of volumetric moisture content (W_v), about 1%-in the range 10-25% W_v, and about 1,5% for the higher moisture contents.

2. Designing of mobile specialized machines, which combine the drilling of bore holes or penetration with investigations by means of radioisotopes. For example, we point out at two types of such machines:

a) type of "Penekar" (elaborated by Institute for Engineering geology and Hydrogeology) by means of which the density and moisture content measurements are carried out during the penetration process.
b) more light-weight machine of type of "Fuzonkar", by means of which the radioisotope measurements are carried out in the hollow tubes after their plunging (or driving in by any way) into the soil to needed depth. Both types of machines are destined for engineering investigations.

3. Development of new methodics, allowing to use the radioisotope gauges in various conditions, and also the new techniques of calibration of density and moisture meters.

Some advances in this activity are achieved as yet only for the neutron moisture meters of depth types and for gamma-density meters of transmission subsurface types.

In reference of neutron moisture meters some methods permitting to transfer from definite conditions of measurements to other conditions, are developed, (for example, from one diameter of bore hole to another).

The techniques eliminating the effect of variable concentrations of some chemical elements (chlorine, iron, manganese) which distort the results of moisture, is developed. (The method of complete calibration of neutron depth probes using the measurements in two soil mediums with different values of volumetric moisture content, and in water, is elaborated. This method is based on the investigation of neutron field in the soil or water medium around the bore hole. The factors making influence on the results of density determination in surface layers of soils (till the depth of 50-60 cm) for the gamma-transmission density meters, are investigated, the ways of eliminating of these factors are developed.

4. Use of radioisotope and nuclear techniques in:

a) engineering;
b) geotechnical control of basement quality.
c) scientific investigations in the construction, e.g. in the investigations of efficiency of various installations and sets at the constructional sites, regime observations during the investigations of slide, consolidation, slumping and deformation processes, etc. One of the ways of further development of radioisotope and nuclear techniques is the use of betatrones, microtrones and neutron generators instead of stationary radioactive sources.
These devices allow to obtain the great flows of irradiation in needed time; in some cases they can be more applicable for investigations. For example, we shall describe the experience of use of a portable betatrone for investigations of soil deformability (Tomsk civil-engineering Institute).

The experiments were carried out in a tray of dimensions 30x30x21 cm, where the lead grains of 4mm, were put in soil over the net 2x2cm. The load on stamp of 25÷50cm^2 area increased up to the ultimate. The soil with marks was examined with brake gamma-irradiation from betatrone. The positions of marks and stamp was fixed on the roentgen film, with exposition time -10-20 min. Dimensions of betatrone are 52x40x40cm, weight-100 kg, power -1,8kw, maximum electron energy- 6 mev.

By comparison of films, it became possible to define the form of soil deformation zones and estimate their sizes. Increase of energy of accelerated electrones to 30-60 mev, will allow to use the betatrone not only for laboratory tests, but for the "in site" experiments.

SOIL CLASSIFICATION WITH STATIC CONE.

Manuel Tapia G. (Venezuela)

The sedimentary soil classification based on the Cone Penetration test ("qc" vs "fs") as used in Holland, Belgium, France and U.S.A. (Florida) has been found suitable to the sedimentary soils of Venezuela.-

Specific values for a classification of residual soils have been developed during a research carried on by myself.- An advanced report over these results I am presenting to you.-

Research On Residuals Soils And Weathered Schists.-

A Dutch Static Cone of 17.5 tons, penetration capacity carrying a friction jacket cone (Begeman) was used in more than 30 soundings, each of then coupled with Standard Penetration Tests.-

These borings were performed in the residual souls and weathered schists existing in Caracas and Valencia, cities of Venezuela.-

The results are exposed in the table No.1-

It is remarkable that the positive or negative friction acting on the piles can be calculated with the values of local friction.

A comparison with the sedimentary soils show that the residual soils have local friction of higher values.-

Residual Soil (SfM) have:

fs= 0.40 to 3.0 Kg/cm^2, and a the sedimentary silty sands have:

fs $<$ 1.00 Kg/cm^2.-

The weathered schists also show local friction of 3.0 to 12.0 Kg/cm^2.

These higher values of local friction could be originated by the foliated structure of the parent schists.-

Notice that the readings of local lateral friction are 50% higher than their actual values in sands, gravels, residual soils and weathered schists.-

We emphatically recomend to use the Static Cone Penetration Test as a means to classify the sedimentary and residual soils.-

The characteristics of weathered schist are also determinable with the Static Cone Test.-

Table No. 1

CLASSICIATION OF RESIDUAL SOILS

Soil Type		qc, Kg/cm^2	fs, Kg/cm^2
Residual Soils	Loose silty fine sand sometimes with scarse organic metter (SfM)	8 to 50	0.40 to 1.00
	Loose clayey fine sand (SfC)	8 to 50	1.00 to 3.00
	Medium dense silty fine sand sometimes with scarse graphitic material.- (SfM)	50 to 100	0.40 to 3.00
	Medium clayey fine sand (SfC)	50 to 100	3.00
Ultra-weathered Schist	Talc-micaceous schist (Silty sand, texture)	100 to 120	6.00 to 12.00
	Micaceous-schist (Silty sand or clayey sand, texture)	100 to 400	3.00 to 12.00
Weathered Schist	Quartz-schist (Silty sand, texture)	> 500	-

Results in The Fills Over the Studied Soils

Young Fills	Mainly loose talc-schist material	5 to 20	0.20 to 100
Ancient Fills	Graphitic-micaceous-schist.-Loose or medium dense material	20 to 60	0.20 to 1.00
	Micaceous and graphitic schist.-Medium dense material.	60 to 120	1.00 to 3.00 With occurrence of quartz $\geq$ 3.00
	Quarzt schist.- Dense material	120 to 200	1.00 to 6.00

AN APPARATUS FOR MEASURING OF RESILIENT SETTLEMENTS. V.P.Titov, V.I.Khromov (USSR)

The apparatus was developed and tested on a number of the operating railroads. The use of this apparatus helps to define elasticaly deformed consistence of a subgrade under different types of rolling stock in real conditions, what is significant step for investigation of undesirable effect on ground because of traffic.

The apparatus (Fig.1) is column 1 placed in boring hole and fastened there by means of its end 2. A cross section of column is tube. The elastic elements 3 supplied with tensoresistors are plased in cross holes drilled in column by certain intervals along it. The special anchors (Fig.1b) are jointed to them. The anchor consists of a body 4 and flexion cambered springs 5, which are fastened to its edges in diametrically opposite order. The anchor body is a hollow cylinder which has internal diameter several more external column one.

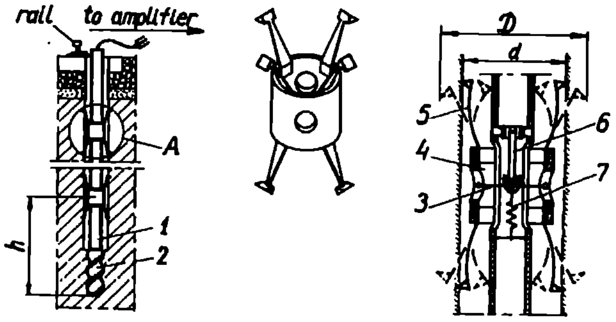

FIG.1. SCHEME OF APPARATUS

In loose consistence external size between anchor spring ends D is more boring hole diameter d (Fig.1c). A support 6 with spring 7 holds the elastic element in a middle. In internal column hallow there are a covered wires, which connect gages to tensometrical apparatus. Besides of main elements above noted there is a mechanism, which regulates a position of the anchor spring ends as it is shown with dotted line in Fig. 1c. It is operated by means of wire cords, which pass along the column. In present figures this device is not shown.

When the apparatus is used a sequence of operation is such. The apparatus is plased in boring hole and driven in its end. When it is in process the ends of anchor springs are deflected to surface of column (Fig.1b, dotted lines). Then, they are released by means of mechanism and the anchors are engaged with surrounding ground (Fig.1b, continuous lines). The wires of gages are connected to the apparatus.

When transport acts on a track each layer of subgrade h between the anchor and column end is compressed and the anchor is displaced relatively column. Obviously, that in this case the deflection of elastic element is equal to the displacement of anchor. A signals pass of sensitive tensoresistors to apparature and are fixed by oscillograph tape.

After the measurement was finished the ends of anchor springs is defelected to column by means of mechanism noted above and apparatus is elevated from boring hole.

Contemporary use of number of the apparatus displaced in certain order in a cross section of subgrade gives a distribution of resilient settlement in this section when cars pass by it.

A real values of settlements were obtained by means of this oscillograms and calibraiting coefficients for each elastic element. As experience shown an accuracy of measurement equal 0,05 mm could be reached by means of use of this apparatus and standart tensometrical apparature. Fig.2 shows maximum values of resilient settlements which occured in observed cross section of subgrade (embankment) when burdened and empty cars passed by it.

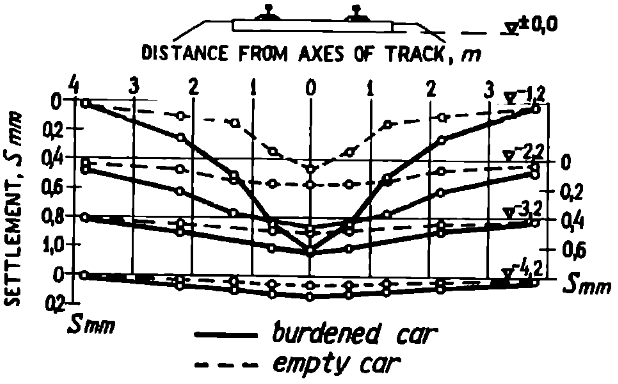

FIG.2. DISTRIBUTION OF SETTLEMENTS

A NEW INSTRUMENT FOR MEASURING SETTLEMENTS.

M. Tominaga and Y. Echigo (Japan)

In a steelwork wide and firm ground is required for storing ores, coals and other raw materials to support a load intensity reaching up to 40 tons/m^2 The great weight of such raw materials causes a considerable degree both of settlements and lateral displacements. Needless to mention, an accurate measurement of settlements and lateral displacements has a great significance in design, performance, operation and maintenance of these yards. Settlement plates, settlement bars and continuous settlement gauges have been used conventionally. However, each has disadvantages They are not durable enough to meet an in-situ measuring requirement for two or three years. Furthermore, they are not flexible enough for speedy measurement even if the raw materials are on the yard.

The new instrument introduced in this paper utilizes the principle of the siphon as shown in Fig. 1(a). Amply dependable for its accuracy, it is designed to meet measuring requirement at any level, whether surface or underground (about 20 m). A covered tube type (Fig. 1(a)) will be best for an underground measurement.

Settlements were observed in a test embankment performance with the new instruments shown in Fig. 1(a). Fig.2 shows an example of the observed settlements.

A trial use of the automatically recorded water-pressure gauges turned out to be a success. This method will be put into wide use in the near future. A square-shaped stainless steel tube which was specially manufactured was inserted into the ground as shown in Fig. 1(b). For the measurement of lateral displacements the roller type inclinometer was devised and adopted. Gelatinous asphalt emulsion around the tube in Fig. 1(b) functions both as the fixer of the square tube inserted in the boring hole, 30 cm in diameter, and as the buffer between the square tube and the surrounding soil, so as to adapt itself to lateral displacements of the surrounding soil. Fig. 3 shows an example of the lateral displacements observed by means of this inclinometer.

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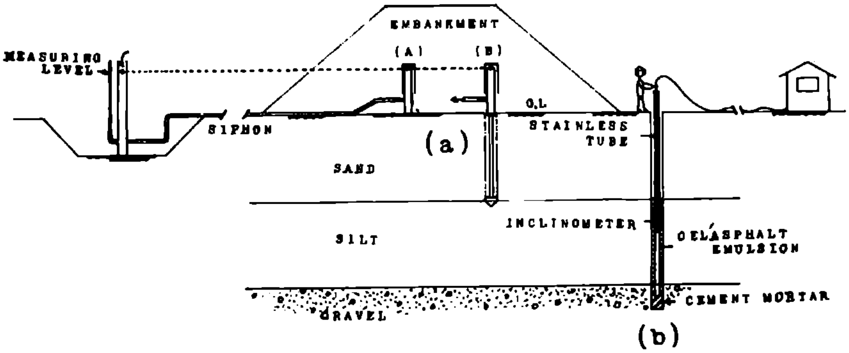

Fig.1 (a) The new Siphon Type Settlement Gauges
(b) The Roller Type Inclinometer

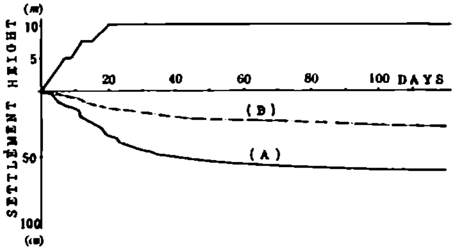

Fig.2 Observed Settlement Curves

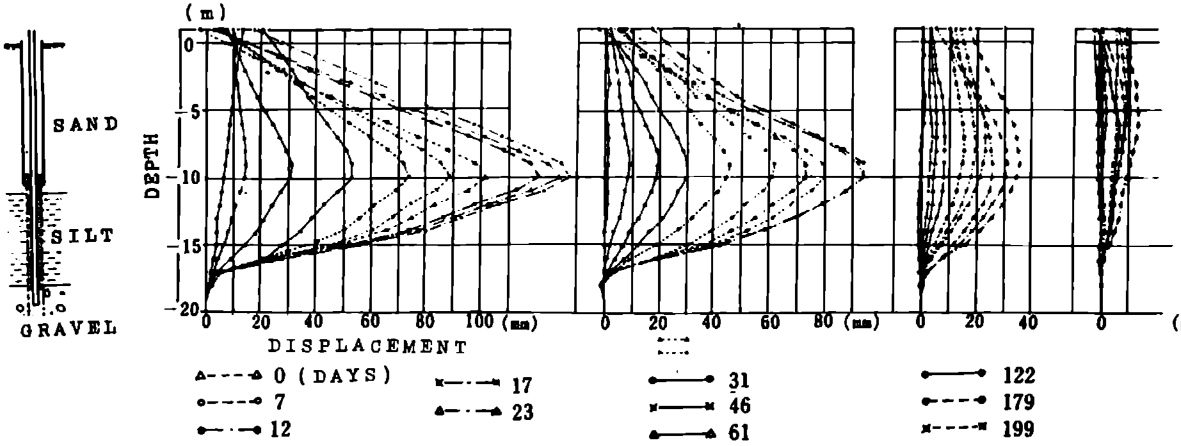

Fig.3 Observed Lateral Displacements below Ground

APPAREILLAGE D'ETUDE DYNAMIQUE DES SOLS FINS ROUTIERS. Verstraeten J., Veverka V., Fagnoul A., Bolle A. (Belgique)

RESUME. Dans le cadre de la mise au point d'une méthode générale de dimensionnement des chaussées, le Centre de Recherches Routières (Belgique) a confié à l'Université de Liège (Professeur A. Fagnoul) l'élaboration d'un appareillage d'études du comportement des sols sous des charges dynamiques.
Le présent article décrit tout d'abord le principe et les buts de cette recherche.
Une éprouvette triaxiale est soumise à une pression hydrostatique constante et à un déviateur d'allure sinusoïdale. Les grandeurs mesurées sont la déformation irréversible, la déformation réversible, en amplitude et en phase, et la teneur en eau. On désire ainsi comparer le comportement dynamique de différents sols et éventuellement mettre au point des essais simples permettant de prévoir ce comportement.
Ensuite, l'appareillage est décrit dans son état actuel, avec une brève évocation des développements ultérieurs envisagés.
Finalement on donne un résultat extrait de la première série d'essais réalisés.

BUTS DE LA RECHERCHE

1. Déterminer de manière quantitative les grandeurs nécessaires pour apprécier la contribution des sols au comportement des chaussées.
    a. Comportement réversible pour le dimensionnement à la rupture.
    b. Comportement irréversible pour le dimensionnement aux déformations permanentes.
2. Comparer les sols entre eux sous l'angle du point (1) ci-dessus.
3. Chercher à mettre au point des essais simples pour juger rapidement du comportement mécanique des sols sur la base des résultats du point (1).

PRINCIPE DES ESSAIS

1. Sols considérés

Les essais portent actuellement sur un sol non-cohésif (sable de Mont-Saint-Guibert). Ils seront ensuite étendus à d'autres sables caractéristiques, puis l'étude envisagera des sols cohésifs, principalement des limons, rencontrés habituellement en constructions routières en Belgique.

2. Conditions d'essai

a. Les sols sont définis par leurs caractéristiques géotechniques : porosité, degré de saturation,...

b. Les conditions de sollicitations sont les suivantes : après une consolidation sous les pressions minima d'essai, on réalise le cisaillement triaxial (actuellement drainé) avec les contraintes suivantes :

$$\sigma_2 = \sigma_3 = \text{Constante},$$

$$\sigma_1 = \text{Constante + fonction sinusoïdale (actuellement); fréquence = 5 Hz.}$$

L'essai est poussé jusqu'à 10^6 cycles ou limité à une déformation verticale de 20 %.

3. Mesures effectuées

a. Avant et après l'essai proprement dit : détermination de p, w,...

b. En cours d'essai, mesure du tassement vertical

- irréversible,
- réversible (en amplitude et en phase).

Ces mesures permettent de déterminer l'évolution de la déformation permanente $\bar{\varepsilon}$ et du module complexe E^*, en fonction du nombre de répétitions N.

c. Mesure de la variation du volume d'eau dans l'éprouvette. Cette mesure nous donne une estimation valable de la variation du volume de l'éprouvette dans le cas particulier des sols non-cohésifs saturés. Pour le cas des sols non-saturés, l'appareillage de mesure des variations de volume doit encore être conçu, ainsi que l'appareil de mesure de la pression interstitielle pour les sols cohésifs.

APPAREILLAGE

L'appareillage est installé dans un local climatisé. Ceci permet d'obtenir une excellente stabilité de tout le matériel électronique. De plus, cela permet de limiter les erreurs lors des mesures des variations de volume d'eau.

1. Application des sollicitations

a. Eprouvette et cellule

Les essais sont réalisés sur des éprouvettes cylindriques (ϕ = 50,8 mm, h = 100 mm) enrobées d'une membrane de caoutchouc et placées dans une cellule triaxiale. Le drainage se fait par des pierres poreuses supérieure et inférieure.

b. Pression latérale

La pression latérale est appliquée par l'eau contenue dans la cellule qui est en communication avec un réservoir soumis à une pression d'azote réglée par un détendeur de précision.

c. Mise en charge verticale

Une vanne électromagnétique alimentée par de l'huile sous une pression de 100 bars actionne un vérin à double effet. Celui-ci applique l'effort à l'éprouvette par l'intermédiaire d'un élément souple et d'un capteur d'effort placé à l'intérieur même de la cellule. Ce capteur fournit une mesure de la force réellement appliquée à l'éprouvette et cette mesure alimente en retour la commande d'asservissement de la vanne. Cette solution élimine le problème de frottement à l'entrée de la cellule et permettra peut-être de mesurer de manière assez simple les variations de volume de l'éprouvette, même dans le cas de sols non-saturés. L'élément souple permet de rendre le système indépendant des variations des caractéristiques mécaniques des éprouvettes.

d. Logique de commande

Le démarrage d'un essai comprenant l'application simultanée pour une durée prédéterminée de la pression latérale et de la pression verticale de consolidation ainsi que le début du cisaillement sous charges répétées, commençant au point bas de la sinusoïde, est entièrement automatisé, dans le but de réaliser des conditions d'essai le plus reproductibles possible.

e. Performances

En restant dans des limites acceptables au point de vue de la précision d'application et de mesure des efforts, il est possible d'appliquer des déviateurs verticaux ($\sigma_1 - \sigma_3$) de forme sinusoïdale ou triangulaire variables entre 0,050 kgf/cm^2 et 5 kgf/cm^2 avec une erreur de ± 0,002 kgf/cm^2 dans le domaine de fréquence de 0 à 10 Hz.

2. Equipement de mesure

a. Mesure du déviateur

La mesure de la force verticale alimente la chaîne d'asservissement et est également introduite dans les circuits de mesure pour la détermination du module complexe.

b. Mesure du tassement

Le tassement de l'éprouvette est mesuré par l'enfoncement du piston dans la cellule à l'aide de capteurs inductifs de déplacement. La précision nécessaire pour la mesure du tassement réversible imposant l'utilisation d'un capteur de course très réduite (± 0,6 mm), il a été nécessaire de rendre celui-ci solidaire d'un support mobile qui le maintient en permanence à l'intérieur de son domaine linéaire de mesure. Le déplacement de ce support mobile, entièrement automatique, est mesuré par un capteur de course plus longue (± 12,5 mm) et le tassement est calculé à partir de ces deux mesures. La précision obtenue est de 5.10^{-4} mm pour la déformation réversible et 10^{-2} mm pour le tassement total.
Il est cependant nécessaire de tenir compte des déformations du capteur d'effort placé à l'intérieur de la cellule.

c. Pression latérale

La pression latérale appliquée est contrôlée par un manomètre à mercure. Cependant, dans le but d'étudier les variations de pression induites dans un système fermé par la variation du déviateur, un capteur de pression à jauges ohmiques, raccordé sur la cellule et connecte aux circuits de mesure permet de mesurer la pression latérale dans la cellule isolée.

d. Variation de volume d'eau

Un réservoir cylindrique relié aux circuits de drainage permet, grâce à un flotteur dont le déplacement est mesuré par un capteur inductif, d'obtenir la mesure des variations du volume d'eau de l'éprouvette avec une précision de 0,05 cm^3. Cette précision est obtenue grâce aux très faibles variations de température (± 0,1°C) réalisées par la climatisation.

e. Contrôles

Le contrôle des divers paramètres de l'essai et du bon fonctionnement de l'installation est assuré, soit par des circuits automatiques, soit visuellement pour la partie haute pression. Un dépassement des limites préréglées entraîne soit le déclenchement et l'arrêt de l'essai en cours, soit encore une correction automatique comme dans le cas du support mobile du capteur de déplacement.

f. Affichage des paramètres

Un oscilloscope à double trace permet le contrôle de la forme de la mise en charge et du bon fonctionnement de la chaîne d'asservissement. Il permet également la visualisation de la déformation verticale réversible et de la fonction $(\tilde{\sigma},\tilde{\varepsilon})$.

Divers ampèremètres connectés directement aux appareils de mesure ou raccordés à l'ordinateur fournissent à tout instant e.a. la valeur de la pression latérale, le tassement total irréversible (valeur calculée), etc,...

Un traceur de courbes X-Y sur papier permet à l'ordinateur de tracer en fin d'essai les divers résultats sous la forme désirée.

Un compteur électromécanique permet de visualiser le nombre de cycles révolus. Il ne joue cependant aucun rôle actif dans le système de mesure.

g. Développements ultérieurs

Outre la mise au point d'un système de mesure des variations de volume valable même pour les éprouvettes non-saturées, il faudra également envisager la mesure de la pression interstitielle dans les sols cohésifs.
D'autre part, une automatisation plus poussée du point de vue contrôle et sécurité afin de permettre à l'appareillage de fonctionner sans surveillance et en toute sécurité, ce qui est indispensable pour les essais de longue durée.

3. Traitement digital par ordinateur intégré

a. Présentation du matériel

Lors de l'étude des systèmes de mesure, il est apparu assez rapidement que la complexité des mesures à réaliser ainsi que la fréquence de celles-ci rendaient impossible la lecture directe. Les enregistrements oscillographiques U.V. sur papier photosensible réalisés au début manquaient de précision, ne permettaient pas la mesure du module complexe et requerraient la présence constante d'un opérateur. La solution idéale était l'intégration au système d'un petit ordinateur connecté à des appareils de prise de mesure et permettant de commander, en temps réel, des processus extérieurs. Nous utilisons donc actuellement un ordinateur Hewlett-Packard de 8 K mots de 16 bits de mémoire centrale, connecté à une base de temps programmable, à un voltmètre digital à 8 canaux, à une sortie double "digital-analog", à une sortie "relais", ainsi qu'à un lecteur optique de bandes perforées et à un téléscripteur. Ce système permet le contrôle continuel du bon fonctionnement de l'appareillage, la commande du processus de démarrage, la prise des différentes mesures, leur traitement et leur stockage, le tracé des courbes obtenues, etc,... La souplesse de ce système permet de modifier à volonté le processus.

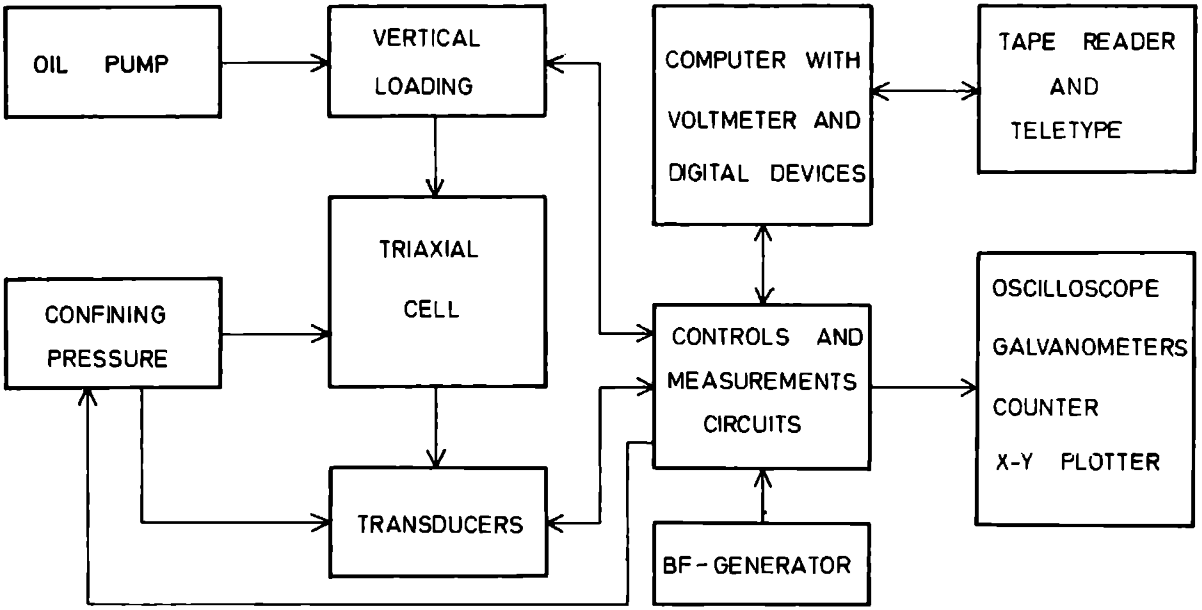

Figure 1. Appareillage.- Apparatus.

b. Procédés de mesure et traitement

Les signaux fournis par les divers capteurs sont amplifiés, si nécessaire, et sont mesurés par le voltmètre digital à 8 canaux. La fréquence de commutation et de conversion de ce voltmètre (35 KHz) est telle qu'elle permet la mesure d'un nombre suffisant de points répartis sur un seul cycle de chargement. La rapidité de l'ordinateur permet d'effectuer simultanément ces mesures et de calculer ensuite, sans interruption dans les lectures, les diverses valeurs désirées. De plus, il surveille simultanément certains paramètres et intervient au besoin directement pour les corriger. Après l'essai, l'ordinateur est disponible dans les limites de sa capacité de mémoire pour effectuer tout traitement désiré des résultats.

c. Développements ultérieurs

Dans le stade actuel, la sortie "digital-analog", qui fournit deux tensions électriques générées par l'ordinateur, et la sortie "relais" qui fournit 16 contacts indépendants, ne servent qu'à l'affichage de certains paramètres, au tracé des courbes et aux interventions de contrôle ou de correction. Elles permettront, dans un stade ultérieur, de réaliser des cycles de chargement plus complexes (amplitude variable, formes non-sinusoïdales, temps de repos entre les cycles, etc,...). De plus, la souplesse de ce matériel est telle qu'un remaniement complet du processus d'essai ne nécessiterait qu'un effort de programmation, sans modification sensible de l'appareillage.

EXEMPLE DE RESULTAT

Les conditions d'essai sont :

$p = 0{,}40$
$S_Y = 1$
$\sigma_2 = \sigma_3 = 0{,}050$ kgf/cm^2
$\sigma_1 = 0{,}100 - 0{,}400$ kgf/cm^2
Fréquence = 5 Hz

A la figure 2 sont représentés en fonction du nombre de cycles N :

- le tassement irréversible $\bar{\varepsilon}$,
- le module complexe E^* ,
- le déphasage $(\bar{\sigma},\bar{\varepsilon})$.

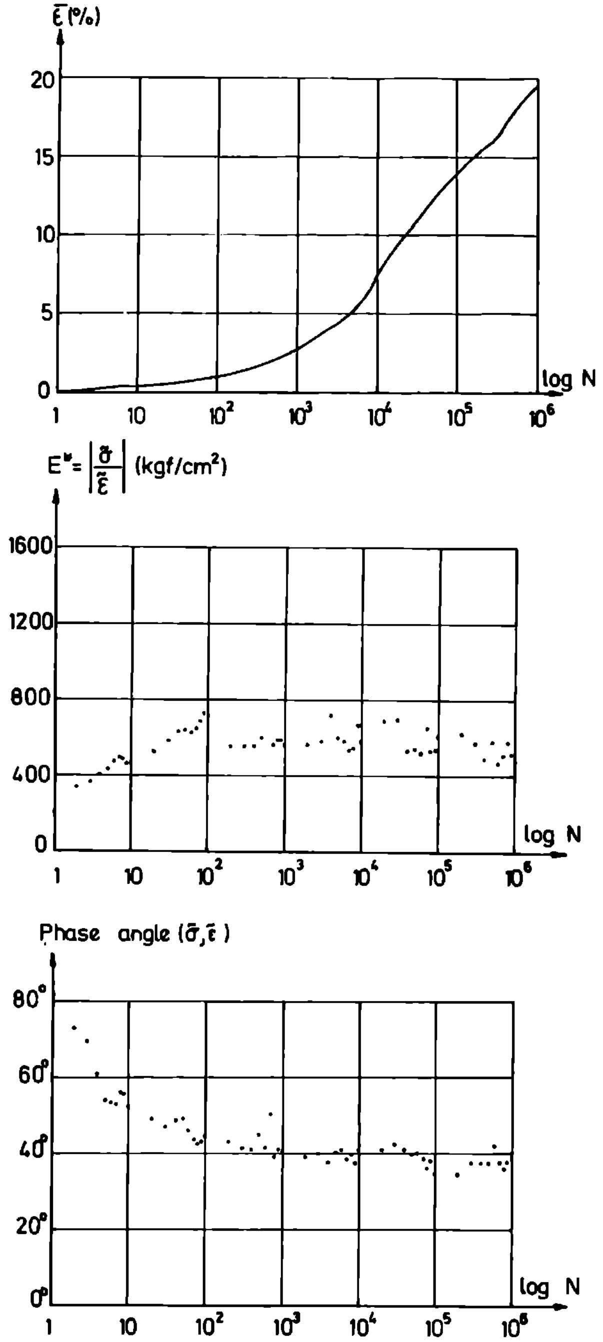

Figure 2. Résultats.-Results.