Proceedings of the Fifth Canadian Soil Mechanics Conference, January 10 and 11, 1952

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https://doi.org/10.4224/40000388
Technical Memorandum (National Research Council of Canada. Associate Committee on Soil and Snow Mechanics); no. 23, 1952-05

The Associate Committee on Soil and Snow Mechanics is one of about thirty special committees which assist the National Research Council in its work. Formed in 1945 to deal with an urgent wartime problem involving soil and snow, the Committee is now performing its intended task of co-ordinating Canadian research studies concerned with the physical and mechanical properties of the terrain of the Dominion. It does this through subcommittees on Snow and Ice, Soil Mechanics, Muskeg, and Permafrost. The Committee, which consists of about fifteen Canadians appointed as individuals and not as representatives, each for a 3-year term, has funds available to it for making research grants for work in its fields of interest. Inquiries will be welcomed and should be addressed to: The Secretary, Associate Committee on Soil and Snow Mechanics, c/o The Division of Building Research, National Research Council, Ottawa, Canada.

NATIONAL RESEARCH COUNCIL

CANADA

ASSOCIATE COMMITTEE ON SOIL AND SNOW MECHANICS

TECHNICAL MEMORANDUM NO. 23

PROCEEDINGS OF THE

FIFTH CANADIAN SOIL MECHANICS CONFERENCE

JANUARY 10 and 11, 1952

Ottawa
May, 1952

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PROCEEDINGS OF THE FIFTH ANNUAL CANADIAN SOIL MECHANICS CONFERENCE

Foreword

This is the record of a Conference of most of the active Canadian workers in the field of soil mechanics, held in the Seminar Room of the Montreal Road Laboratories of the National Research Council of Canada, in Ottawa on January 10 and 11, 1952, under the auspices of the Associate Committee on Soil and Snow Mechanics. About 70 persons were present from various parts of Canada to take part in general discussions relating to problems in soil mechanics and to hear talks on allied subjects. A list of those attending is included as Appendix A of these Proceedings.

As proposed at the start of the Conference, the first day was devoted to the presentation of papers and reports while the second day was reserved for discussion on problems of general interest and business matters of the Conference. The material contained in Section 2 to Section 7 was presented on January 10 and the remainder on January 11.

SESSION OF JANUARY 10, 1952

Section 1

Introductory Remarks

by

R. F. Legget

Mr. Legget served as Chairman to open the meeting. He welcomed the delegates to the Fifth Annual Canadian Soil Mechanics Conference and outlined the character and procedure of these meetings.

He mentioned that the conferences were sponsored by the Associate Committee on Soil and Snow Mechanics of the National Research Council, to encourage discussion and exchange of ideas in the field of soil mechanics and foundation engineering.

Although this was the fourth such meeting held in Ottawa, he hoped that future gatherings might be held elsewhere according to the wishes of the delegates. He looked forward to the next meeting which might take place in Ottawa since in all likelihood it would be possible to have this meeting in the new building for the Division of Building Research, at present under construction. He said that since these meetings were primarily intended to be technical and time was limited, he would open the meeting by calling on Mr. Schriever to present the first prepared talk.

Section 2

Toronto Subway Research

by

W. R. Schriever

Mr. Schriever's talk was divided into three sections: first, the necessity for and layout of the subway; second, construction procedure; and third, research on the construction project.

Necessity and Layout

Toronto is situated on the north shore of Lake Ontario with its business section near the lake. Owing to its location the City extends north, east and west to a greater extent than a city of comparable size not restricted in this way. The traffic flow is mainly in two directions, parallel and at right angles to the shore of the lake. The greatest need for a subway is in the direction of greatest traffic flow combined with greatest congestion, i.e. at right angles to the lake and therefore, of the two subways planned at the present time, the one in this direction along Yonge Street is being built first. It can be divided into three main sections corresponding to land values, of which the first in the business part of the City is built directly under Yonge Street, the second on a private right-of-way off Yonge St. but underground, and the third in open cuts with city streets crossing the subway on bridges.

Construction Procedure

The question has often been asked why the subway was not tunnelled without disrupting street traffic in the business district of the city. In order to facilitate the transfer of passengers from the surface transportation system to the subway and vice versa, it is desirable to construct a subway as shallow as possible and therefore tunnelling is impracticable. The construction method in which a subway is built from the surface in a deep cut covered by a temporary street deck is known as the cut-and-cover method, and this was used in Toronto. It consists of driving steel H-piles into the ground along both sides of the street at 6- to 8-foot intervals. These so-called soldier piles which are driven below subgrade level serve to support the sides of the excavation as well as the vertical loads of the traffic and the temporary street deck. The street is then excavated between the rows of piles to a depth of approximately 10 feet and a temporary deck consisting of a combination of steel and timber beams is placed over the excavation. Normal street traffic is resumed and further excavation as well as the construction of the actual subway structure is carried out underneath. Later the space above the reinforced concrete structure is backfilled with sand, the street deck removed, and pavement laid.

Apart from a small area in the business section, where bedrock is encountered at depths from 20 to 40 feet, the subway is founded in soils of various types. Glacial tills and interglacial sands and silts are predominant.

Research on the Construction Project

As the construction of the subway is providing an unusual opportunity to investigate soil and foundation conditions along a continuous section through the City of Toronto and to study some related design and construction problems, it was decided to use this project as an "outdoor laboratory" for such investigations. The Division of Building Research of the National Research Council therefore approached the Toronto Transportation Commission and a co-operative research project was arranged. (The speaker expressed his appreciation for the assistance given by the Toronto Transportation Commission in conducting this research.)

The main research investigations are as follows:

(1) Investigation of soil conditions and recording of the complete soil profile along the subway;

(2) Measurement of stresses in deck beams and other temporary steel structures;

(3) Investigation of vibrations resulting from construction operations;

(4) Measurement of soil temperatures.

In addition, several other smaller investigations were carried out and assistance given to the Toronto Transportation Commission in some special construction problems.

The soil profile is being taken by recording soil sections at 50-foot intervals, both from an engineering and a geological point of view. The geological work is done in co-operation with a committee formed of geologists interested in the Pleistocene geology of the area from the University and several government departments. The soils are described in detail and the information summarized on drawings (of which an example was shown by the speaker). It is hoped that this record will form a start on the collection of local soil records in Toronto. The soil profile corroborated the information obtained from borings carried out before construction started.

During this work, soil samples were taken at frequent intervals and tested at the soils laboratory of the Toronto Transportation Commission for water content, grain size distribution, unconfined compressive strength, Atterberg limits, and other properties in special cases.

Many interesting soil problems were encountered during construction. Among these were the underpinning of buildings along both sides of the subway, the deflection of some of the soldier piles by large boulders contained in the till, the dewatering of part of the excavation by wellpoints and the backfilling of the space between the subway and the street surface.

The speaker illustrated some of the work by showing a number of slides.

Reports on various phases of the research work are in preparation.

Discussion

In reply to a question regarding the procedure followed when backfilling with sand around the completed subway structure, Mr. Schriever said that during construction operations it was desirable that the deck should be left in operation as long as possible. Water was therefore used to compact the sand as it was not possible to use heavy equipment in the restricted space under the deck. The drains of the subway handled most of the excess water.

Mr. Knight asked if the volume of timber in the deck were known. Mr. Schriever said he did not know the exact figure but mentioned that the 12- by 12-inch timbers had been shipped via the Panama Canal. The Chairman said he thought that it would be about 3,000,000 board feet.

Mr. Hall asked if vibrations due to pile driving close to existing buildings were recorded and if any relationship between factors had been established. Mr. Schriever said that a Leet portable seismograph had been used to record vibrations, caused both by pile driving and streetcar traffic. These records had been taken so that a comparison could be made; the results were not yet analysed. In the measurements emphasis was placed on a determination of the rate of decrease of vibration with distance and depth.

The Chairman mentioned that the reports on Mr. Schriever's work would be somewhat delayed as the work was being done for the Toronto Transportation Commission. He said that there was a scarcity of literature available on vibrations due to pile driving but one Belgian paper had been discovered, outlining such work on a tunnel under a river.

Mr. Coates asked about the order of magnitude of the impact factor on the deck beams. Mr. Schriever said that at one intersection the impact factor was around 30 per cent for one vehicle (streetcar), but far less for a combination of vehicles.

Mr. Davis noted that further from the downtown area in Toronto the water table was higher. He asked if this had caused any damage or if the buildings had been underpinned previous to operations. In reply to this, Mr. Schriever said that the water table was not so much higher as more noticeable because the soil was more permeable. He said that settlement due to a lowering of the groundwater table in sandy soils was slight and did not cause any damage. He mentioned that most of the buildings along the subway had been underpinned.

Section 3

Recent Soil Mechanics Studies of the P.F.R.A.

by

R. Peterson

Mr. Peterson said that as the proceedings of earlier conferences contained an outline of the activities and the equipment of the P.F.R.A. Soil Mechanics Division, his report to this meeting would only include an outline of some recent studies that might be of interest.

Proposed South Saskatchewan River Dam

The dam as proposed is an earth fill 210 feet high and containing approximately 35,000,000 cubic yards of fill. The valley at river level is approximately 2000 feet wide and the foundation consists of sand varying from a few feet in depth near the edges of the valley to nearly 100 feet in depth at the centre. This sand in the valley bottom and the thin overburden on the abutments is underlain by soft Bearpaw shale. The river sand and the shale are, therefore, the two major materials to be considered where the dam foundation is concerned.

1. Bearpaw Shale

The Bearpaw shale is of marine origin and is described as a clay shale, although many engineers would be more inclined to regard it as a stiff overconsolidated clay. It has been subjected to pressure of the weight of several thousand feet of sediment and ice in earlier times. The water content varies from 35 per cent at the surface where it has softened, to about 20 per cent in the firm unweathered material. The corresponding range of wet density is 110 to 130 lb. per cubic foot. The Atterberg limits based on air drying are liquid limit of 110 and plastic limit of 20. The shale contains joints and slickensides.

Preliminary studies of this clay shale have revealed many unusual properties and special tests have had to be devised to evaluate these properties.

(a) Liquid Limit Research

It soon became evident that the liquid limit test was very sensitive to the test procedure. In tests conducted at Harvard University it was found that oven drying reduced the value of the liquid limit. On one sample the liquid limit varied as follows depending on the method: air dried, 160; natural water content, 130; and oven drying, 100. This information, coupled with the fact that routine tests indicated variations, led to a research program to determine the most practical procedure to be followed. In this program the following variables have been studied while holding other variables constant: different operators, different machines, length of time of soaking, ratio of amount of soaking water to weight of soil used, and size of particles before soaking. As a result of this study the tentative procedure is to air dry all shale to a moisture content of 4 to 6 per cent whereupon it is ground, passed through a No. 40 sieve, and soaked for a period of 24 hours. Several of the tests indicated that the variations were due to incomplete soaking of the individual particles. It is therefore proposed to extend the soaking period and to standardize the grinding procedure in an attempt to obtain more consistent results.

In conjunction with this program the swell test which is being used by the Bureau of Reclamation was studied. A fairly good correlation was obtained between the results of the swell test and the liquid limit but it was felt that the swell test required almost as much time and care as the liquid limit test. It was therefore decided to continue using the limit test.

(b) Consolidation Tests

Consolidation and swelling tests which have been carried out to determine the characteristics of the shale indicated very high swelling pressures. It was also evident from these that the secondary time effect was very great in comparison with the primary. In view of this, special long time consolidation tests are being carried out. In one test a 4-inch diameter sample, 1 inch thick, was subjected to overburden pressure for a period of several weeks and then subjected to a stress approximately equal to the weight of the dam. It is proposed to leave this stress on the sample for a period of at least one year. A second sample of the same size has been set up and load increments are being added at intervals of two weeks.

(c) Shear Tests

The shear strength of the shale, based on laboratory tests, appears to be considerably greater than the shear strength computed from actual slides although the assumptions as to the depth of the sliding surfaces may be in error. The shale tends to soften at the surface and the presence of several zones of consistency has been established, that is, soft, medium and hard from the surface downward. If the sliding planes occur mainly through the soft surface shales, a better correlation exists between laboratory strengths and strengths indicated by stability studies. Laboratory tests also give reduced strengths with increased time of loading when the samples are held at constant water content.

(d) Slides

In view of the doubtful value of laboratory shear tests in assessing stability problems, it has been decided to study natural slopes and particularly those where movement has occurred. Based on experience to date, a study of aerial photographs is by far the best method of locating the boundaries of active slide areas. In addition, it is necessary to locate the slide surface and to study the material both above and below the slide surface. This has been done by means of bore holes, deep test pits and a test drift. Surprising as it may seem it is often extremely difficult to locate the surface of movement, particularly in material such as this which is jointed and slickensided. A careful study of water content profiles with determinations at intervals of 3 to 6 inches appears to be useful in locating slide surface. In an area where no sliding has occurred, the water content generally decreases uniformly with depth below the weathered zone. Where sliding has occurred there generally is a very abrupt reduction in the water content profile below the sliding surface. This would seem to indicate that the material immediately above the slide surface has been reworked with the result that the water content has increased. In addition to the water content profile the consistency index has also been found to be very useful.

(e) Deep Test Pits

Test pits up to 160 feet deep have been utilized in connection with studies of the shale for purposes of examining the material in place and obtaining large undisturbed samples for testing. It has been found that safety precautions are extremely important when working to this great depth.

The first experience with large diameter holes in the Bearpaw shale involved drilling two 32-inch diameter holes with a well-boring machine. These holes were put down to depths of approximately 100 feet and were cribbed only through the overburden, because it was extremely difficult to carry the cribbing through the shale and at the same time withdraw the auger bucket. This worked very well at first but drying soon occurred in the upper layers of the shale and chunks began to fall from the walls of the bore hole. On the advice of the consulting geologist, these two holes were closed up and declared unsafe. The geologist also recommended that for any future borings the P.F.R.A. work closely with the Provincial Mines Department. The Mines Branch has been most co-operative in working with the P.F.R.A. to develop a safe and practical method for excavation of deep test pits.

The present method is to use wooden cribbing through the overburden. This cribbing generally consists of 3-inch material and the outside dimensions are either 3 by 5 feet or 4 by 4 feet. Starting at the surface of the shale, 48-inch diameter 14-gauge liner plate is used. Each liner plate ring consists of four segments, the width of each ring being 18 inches. Alternately with the liner plate rings, 18-inch spacer bars backed with wire mesh are used. This makes it possible to examine the material through the wire mesh and where further samples are required, the mesh can be cut away and samples obtained. Both the wooden crib portion and the liner plate portion of the pit are divided into two sections by vertical planking. One section is used as a hoisting compartment and the other section is used for access ladders. One very important requirement of the Mines Department was to limit the length of ladder to about 8 feet, successive ladders to be separated by landings, thus making it impossible for a person to fall more than the distance between landings.

It has been found desirable to erect shacks approximately 10 by 12 feet over all pits for protection against the elements. At each pit head a lighting plant and hoisting equipment are utilized. The lighting plant serves also to supply power for an electric fan which provides ventilation to the bottom of the pit. Lighting and hoisting units are both gasoline driven and, in compliance with regulations, are set approximately 50 feet from the pit head to avoid the possibility of monoxide gas entering the shaft.

This method of exploration has proved to be extremely satisfactory and while the cost is admittedly high, it provides an excellent opportunity to examine the material in place. The only difficulty encountered to date has been where seepage water occurs on the contact or in the weathered zone of the shale. Attempts are being made to develop a satisfactory seal. All excavation to date has been by hand methods although air tools would increase the excavation rate.

(f) Pressure Test Section

The Proceedings of the 1950 Soil Mechanics Conference, Technical Memorandum No. 19, contain a brief description of the test drift in the shale at this damsite and mention is made of a pressure section to measure the vertical and horizontal force exerted on a continuous lining by the shale. The details of the pressure test section and the test drift are contained in P.F.R.A. report entitled "Geological Test Drift, Damsite No. 10" December, 1951. The following covers a brief description of the pressure test section along with the readings which have been recorded to date.

The pressure test section is located at the end of the drift and consists of a 20-foot length lined with precast concrete slabs. The slabs are 2 feet wide, 8 inches thick and approximately 6 feet long. The horizontal and vertical slabs are staggered so that the joint between two horizontal slabs falls at the centre of a vertical slab. Each pair of opposite slabs are held in place by pipe struts provided with a screw jack. A pair of horizontal struts are used to hold two opposite vertical slabs and a pair of vertical struts to hold two opposite horizontal slabs. The excavation and the placing of the slabs was carried on continuously and it was therefore possible to make the entire installation in a period of 10 days. Dry packed grout was used to backfill the irregular space between the back of the slabs and the mined surface of the shale. When the installation was completed a predetermined pressure was applied through the screw jacks onto the slabs. Initially a load approximately equal to 130 per cent of the overburden stress (overburden above drift, 64 feet) was applied to both the horizontal and vertical slabs. The screw jacks were left unchanged and load readings taken periodically. The load tended to drop off on both the vertical and horizontal slabs for a period of about three months and then began to increase very slightly. The present load on the vertical struts is approximately equal to overburden and the present load on the horizontal struts is approximately equal to the initial applied load. It would therefore appear as was originally suspected, that the horizontal stress in this heavily preconsolidated shale is greater than the vertical and that such a stress distribution might be expected upon a tunnel lining.

The load in each strut is determined by means of strain measurements utilizing a 20-inch Berry strain gauge. Initially all 40 struts were calibrated in the laboratory and stress-strain curves determined. However, during the course of the readings it has become evident that there are some errors in this system, believed at present to be less than 10 per cent. In the near future, however, it is proposed to recheck the calibration of each strut by temporarily reducing the load to zero. It will then be possible to appraise the accuracy of the method being used.

2. River Sand

Preliminary sampling indicated that the river sand was fine to medium grained with a D_{10} of 0.12 mm., a D_{60} of 0.30 mm. and a coefficient of uniformity of 2.5. The majority of these disturbed samples were recovered by means of a bailer and it was felt that the grain size distribution was reasonably accurate, based on a few tube samples taken as a check. However, no information on stratification or density was obtained.

(a) Cone Bearing Tests

In order to obtain some information on the relative density of the sand in place, a bearing cone (described in "Soil Mechanics in Engineering Practice" by Terzaghi and Peck, Figure 121c, Page 276) was used. This cone was calibrated in the laboratory and also in a sand deposit above the water table in the field. The purpose of the calibration was to obtain an approximate relation between resistance to penetration and density of the sand. Following this, tests were carried out in the river sand which indicated that the sand was at a density near the boundary between the loose and medium dense states.

(b) Undisturbed Sand Samples

Undisturbed samples of the sand were desired to determine the density and observe stratification. However, it was recognized that this would be extremely difficult because the material is below the water table. Some thought was given to using the frozen plug method which had been developed by the Providence District of the U.S. Corps of Engineers. However, this was never carried out because the method is extremely slow and costly.

The U.S. Waterways Experiment Station has recently developed another method for recovering undisturbed sand samples below the water table. This is described in Bulletin No. 35 of the Station entitled "Undisturbed Sand Samples Below the Water Table". It involves drilling a 5-inch diameter hole using rotary equipment and a heavy drilling mud to prevent caving of the hole. Great care is also taken to use slow rotational speeds and avoid loosening or disturbing the material by strong jets of water. The actual samples are recovered using a 3-inch Shelby tube sampler fitted with the standard piston and it is the general practice to recover practically continuous core samples. It has been found that the drilling mud penetrates only a very short distance into the sand. Where it is desired to obtain the density of the sand in place, the sample tubes, which are originally about 36 inches long, are stored in a horizontal position and cut into 3-inch lengths and the sand density in each length determined. Where it is desirable to study stratification the sample tubes are stored in a vertical position and cut longitudinally so that the stratification can be studied in detail. It was found that a metal band saw was the most satisfactory for cutting the tubes. The P.F.R.A. organization has now acquired this method and one hole has already been driven and the samples are now in the process of being examined. While the method is quite slow it appears to be entirely satisfactory.

(c) Pumping Tests

The permeability of the river sand is important in determining the type of seepage control to be used with the dam. A number of pumping tests have therefore been carried out. The earlier tests involved use of an ordinary wellpoint with 80-mesh screen placed at a depth of 4 or 5 feet with eight observation wells placed on two lines at right angles. Following this, a slightly more elaborate test was conducted with a central well 3 inches in diameter extending to a depth of about 20 feet. More recently an 8-inch well extending to a depth of 50 feet with 22 observation wells has been used. The average k for all types of tests was 0.14 foot per minute with no indications of variations with depth.

(d) Steel Sheet Piling Tests

In the design studies of the proposed dam, methods of seepage control that were considered included steel sheet piling, an upstream blanket and a positive trench-type cutoff to shale. The studies revealed that the positive trench type cutoff was very costly, difficult to construct and probably unnecessary. The choice, therefore, had to be made between steel sheet piling and an upstream blanket or both.

In order to obtain more information on the effectiveness of steel sheet piling in controlling seepage, a series of tests were conducted using various sections of sheet piling and typical river sand. The flow of water through the sheet piling interlocks was measured along with the corresponding drop in pressure for various rates of flow. From these test results the reduction in flow due to the sheet pile wall has been predicted. The details of these tests are contained in a P.F.R.A. report entitled "Steel Sheet Piling Studies" December, 1951. On the basis of the studies it was concluded that unless rusting, corrosion, and air locking occurred in the piling beneath the dam, that a relatively short length of blanket was as effective and much more economical than steel sheet piling. The base of the dam as now proposed is approximately 2600 feet wide with a filter drain extending into the fill about 1000 feet from the downstream toe. If steel sheet piling were used it would be placed at about the upstream quarter point. Studies were made to determine the length of blanket which would be equivalent to steel sheet piling placed at this point.

It should be mentioned that several types of arch piling were studied but all behaved in the same general manner from the standpoint of seepage. There was, however, considerable variation depending upon whether the joints were loose or tight. Based on a coefficient of permeability of river sand of 0.14 foot per minute and a head of 190 feet, it was estimated that the seepage beneath the dam with no piling or no blanket would be 31 cubic feet per second. Using the same assumptions mentioned above and sheet piling with the interlocks loose but filled with sand, the seepage was calculated to be 30 cubic feet per second. An upstream blanket 33 feet long would cause a similar reduction in flow. Using the assumptions mentioned above and the interlocks tight and filled with sand, it was calculated that the seepage would be reduced to 28 cubic feet per second. A blanket having a length of about 145 feet would cause a similar reduction. It is quite obvious, therefore, that unless the sheet piling corrodes or air locks, that it is relatively ineffective.

The present proposal is to use a blanket about 1200 feet in length, which according to calculations will reduce the seepage to the order of 18 cubic feet per second. It is felt that natural silting in the reservoir will, within a very short space of time, reduce the seepage to a small fraction of the flow experienced when the dam is initially constructed. It is also proposed to use drainage wells to provide relief at the downstream toe of the dam.

St. Mary Dam

The St. Mary Dam has now been completed and during the past year the reservoir has been almost full for a short period. Some seepage has occurred through bedrock in the abutments and a grouting program to cut this off is nearly completed.

The test apparatus which was installed in this dam has provided very interesting data. From the soil mechanics point of view the impervious central section of the dam, which was constructed of glacial clay having a liquid limit of 35 and a plasticity index of 14, is by far the most interesting. This material was placed in 6-inch lifts and compacted by 12 passes of a sheepsfoot roller exerting unit pressures of about 500 pounds per square inch. A deliberate attempt was made to compact the material slightly on the dry side of the optimum moisture content for field compaction and several per cent below the Proctor optimum in order to avoid the danger of future pore water pressures. Standard Proctor tests indicated an optimum moisture content of 15 per cent at a dry density of 114 lb. per cubic foot. The average dry density as placed was about 110 lb. per cubic foot at a water content of 12 to 14 per cent.

The Bureau of Reclamation type settlement gauges have revealed that the total compression within the impervious section with a height of 200 feet is now of the order of 8 feet and consolidation is still going on at a reduced rate. However, in spite of this relatively high settlement there has been no indication of pore pressure, which is attributable to the fact that the water content during compaction was kept as low as possible without causing serious reduction in density as placed. It is calculated that the density in the lower layers of the fill has now increased to 120 lb. per cubic foot.

Travers Dam

An earth fill dam of comparable volume to the St. Mary Dam has been started on the Little Bow Valley about 40 miles north of Lethbridge. This dam is a unit in the Bow River Development designed to irrigate about 250,000 acres in the area north and west of Medicine Hat, Alberta. The dam is 130 feet high and contains about 4,500,000 cubic yards of fill.

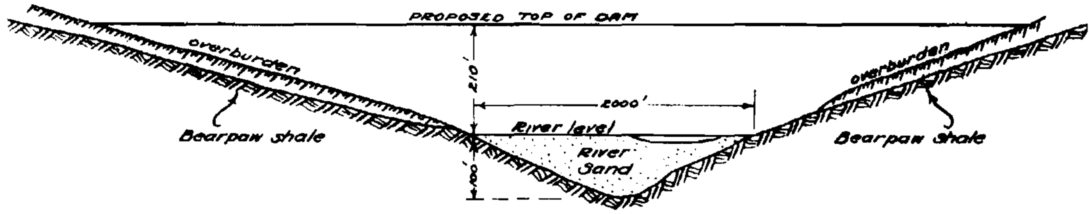
The fill materials for this dam are ideally located. Impervious clay, pervious sand and gravel, rock riprap and concrete aggregate all being available within a one-mile radius of the dam. The foundation, however, consists of Bearpaw shale. In view of the fact that some instability in the shale is evident in one abutment, the fill has been flared out at this abutment and fillets utilized between the dam and the abutment in order to completely stabilize any zones that might tend to become unstable with water in the reservoir. Extensive test apparatus is being installed to record the behaviour of the structure and to obtain further information on the behaviour of the Bearpaw shale.

Canal and Dugout Lining Program

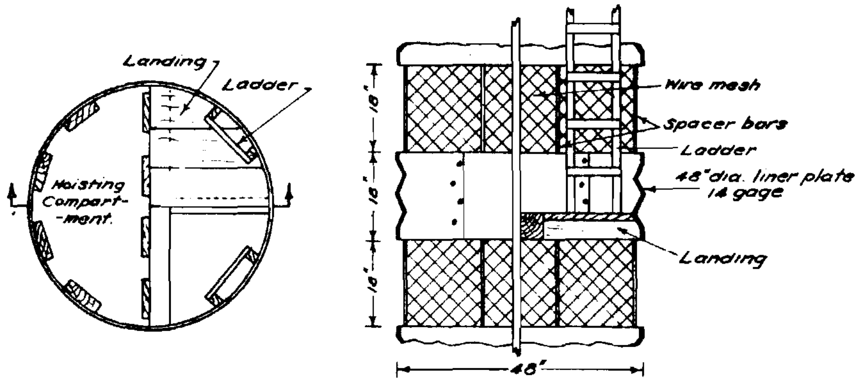
The P.F.R.A. is carrying out an extensive experimental program on canal and dugout lining methods. The First Progress Report covering this investigation was prepared, in March, 1951. Since this Progress Report was prepared, experimental installations of several other types of lining have been tried. These involved bentonitic material sluiced into water, prefabricated asphalt membrane, and catalytically blown asphalt sprayed on to a prepared aggregate base. Based on the studies to date it would appear that where suitable materials are available locally a compacted clay lining about 12 inches thick and covered with about 12 inches of granular material is the most economical.

FIFTH CANADIAN SOIL MECHANICS CONFERENCE
N.R.C. - OTTAWA - JAN. 10-11, 1952
R. Peterson - P.F.R.A.

PROPOSED SOUTH SASK. RIVER DAM



1. Bearpaw Shale - overconsolidated stiff clay
   Water content range - 20% - 35%
   Wet density - 111 - 130 lb. per cu. ft.
   Liquid limit - 110, Plastic limit - 20
   (a) Atterberg limit research - values affected by drying
   (b) Shear tests - may not reflect true strength
   (c) Consolidation tests - secondary time effect great
   (d) Slides - located by air photos
   - slide surface from water content profiles
   (e) Deep test pits - up to 160' - safety precautions very important.



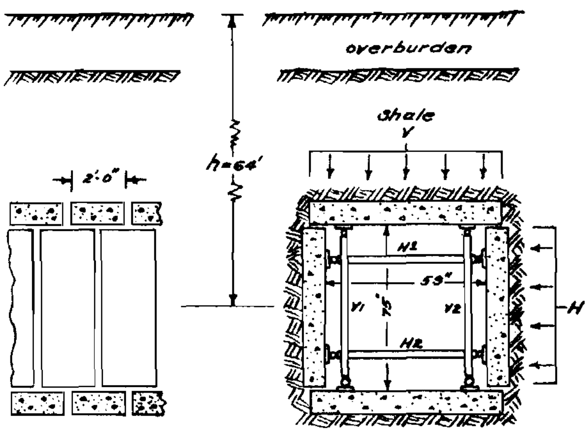
(f) Pressure Test Section - "Geological Test Drift Damsite No. 10" - P.F.R.A. Report April, 1951

γ = soil density

h = depth of cover

A = area of slab

$V_1+V_2 = V$
$H_1+H_2 = H$

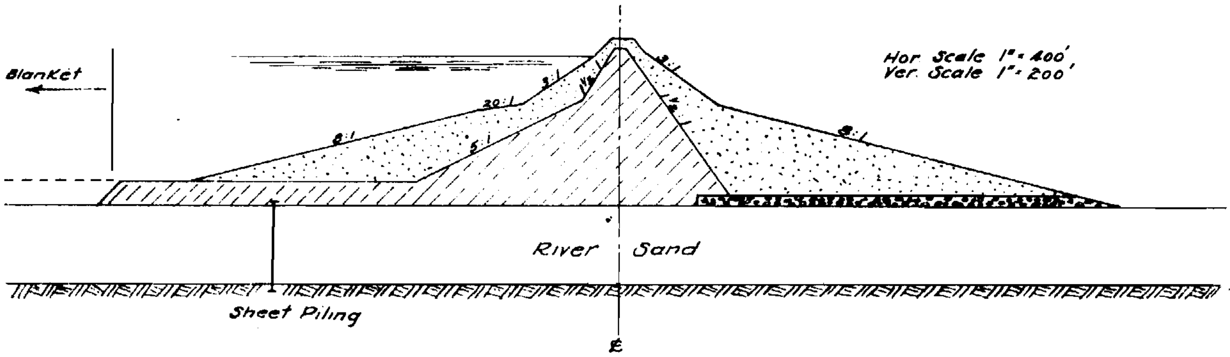


	Unit Stress	
	Vert. = V/A	Horiz = H/A
Initial	$1.26\gamma h$	$1.32\gamma h$
Minimum	$0.99\gamma h$	$1.21\gamma h$
Present	$1.05\gamma h$	$1.34\gamma h$

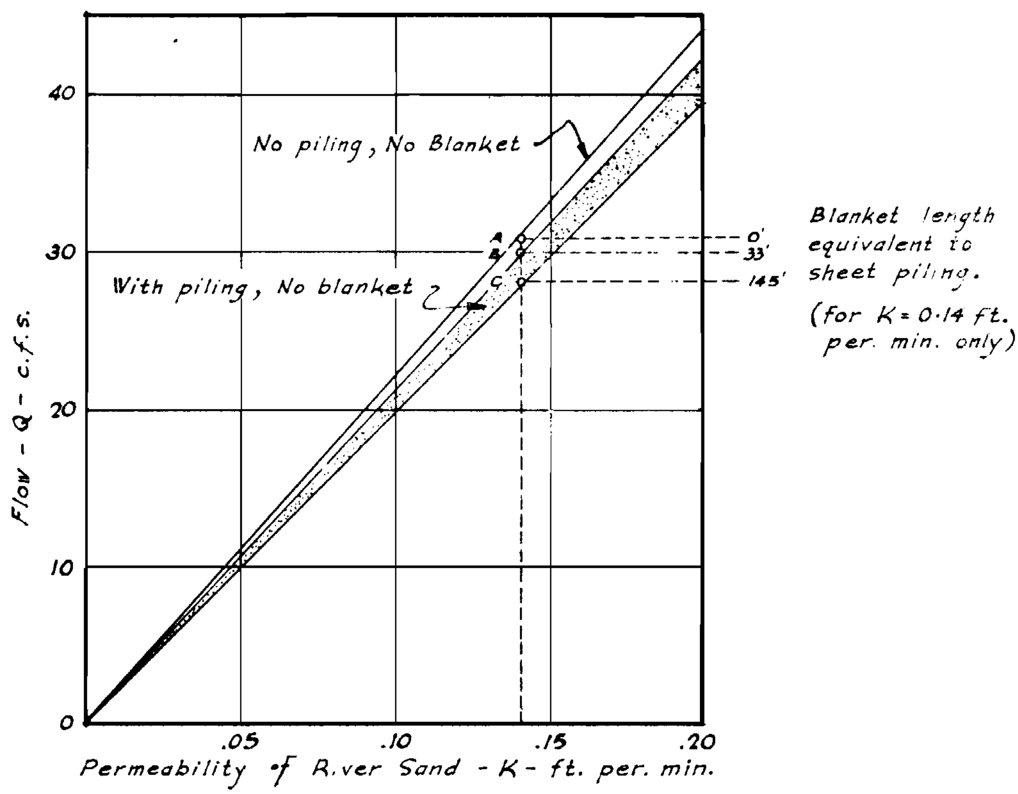
2. River Sand - fine to medium, D_{10} = .12, D_{60} = .30, Cu = 2.5

(a) Cone bearing tests indicated loose to medium density state - see Terzaghi & Peck, Fig. 121c, p. 276.
(b) Undisturbed sand samples - using Vicksburg method - see "Undisturbed Sand Samples below the Water Table", Bulletin No. 35, U.S.W.E.S., Vicksburg, Miss.
(c) Pumping Tests - Average K = 0.14 ft. per min.
(d) Steel Sheet Piling Tests - see "Steel Sheet Piling Studies", P.F.R.A. report, December, 1951.

(i) Cross Section of Proposed Dam



(ii) Flow-Permeability Diagram for above Cross Section



For K = 0.14 ft. per min.:
(A) Q = 31 c.f.s. (no piling, no blanket)
(B) Interlocks loose, sand in interlocks loosely placed, with "k" approx. 0.09 ft. per min.
Q = 30 c.f.s.
Blanket length, equivalent to piling = 33 feet
(C) Interlocks hand-tight, sand in interlocks hand-tamped, with "k" approx. 0.02 ft. per min.
Q = 28 c.f.s.
Blanket length, equivalent to piling = 145 feet

Discussion

Referring to the vertical and horizontal pressures measured in the test drift for the South Saskatchewan River Dam, Dean Hardy commented that he thought it would be misleading to quote them as a percentage of the overburden and thought it should be overburden pressure plus the swelling pressure.

In reply, Mr. Peterson said that because there was very little movement of the shale, it was difficult to segregate between "swelling" and "pressure". The material was not highly swelling but developed high pressures. The measured yield into openings was very low and so the material was not considered to be a swelling shale. The "no yield" pressure was that which was sought in the test drift.

Dean Hardy asked, assuming that the present pressure is constant, if Mr. Peterson expected to get an increase of 34 per cent of the horizontal pressure over the overburden pressure at any depth. Mr. Peterson replied that he thought it would vary to a minor degree.

Regarding the St. Mary Dam, Dean Hardy said that it was interesting to note that 8 feet of settlement had been recorded, in spite of the fact that this was at a smaller pressure than that used in compaction. He wondered if this was because the load was present over a long period of time.

Mr. Peterson said that approximately 6 of the 8 feet of settlement previously mentioned were rapid settlement due to compression of the air voids of the unsaturated soil and took place during construction; the remaining 2 feet of settlement were caused by consolidation owing to air and water escaping from the voids. Mr. Peterson said, in reply to Dean Hardy's question, that they were not considering using lighter rollers.

Mr. Knight asked if sections of the deep test pits had been checked to see if clay was creeping in on the mesh. Mr. Peterson said that most of the "creeping" was due to spalling; no bulging occurred. Mr. Peterson explained that the disturbed material became wet and then moved.

Mr. Coates asked if chemical grouting was economically feasible for creating sand cutoffs. Mr. Peterson said that they were following procedures used in the United States on the Missouri River where the material is similar to that in Saskatchewan and that the possibility of chemical grouting had not been suggested.

Mr. Coates mentioned work that had been done in the San Francisco area to make soils impermeable by chemical means, as reported by Mr. C. H. Lee in the Transactions of the American Society of Civil Engineers, Vol. 106, page 577, in 1941.

Mr. Fairbairn mentioned that timber cutoffs such as Wakefield piling were used in the Province of Quebec and asked if this had been considered at the South Saskatchewan site. Mr. Peterson said that wood piling was not considered since the necessary depths were too great and many boulders were present.

Mr. Peckover asked about the formation of slickensides in the Bearpaw shale. Mr. Peterson said there were various opinions on this. Dr. Terzaghi thought that slickensides were caused by chemical changes in the shale, whereas Dr. Casagrande was inclined to think that they were caused by internal adjustments when the overconsolidating load was removed and the brittle shale had a tendency to pop up.

Section 4

Deep Sounding Methods for Evaluating the Bearing Capacity of Foundations on Soil

by

W. A. Trow

In recent years considerable attention has been given to the use of apparatus for measuring the strength of soil in place. There are good reasons for this attention. It is felt that a more accurate measure of the soil's capacity is obtained by field tests because disturbance of the soil is reduced to a minimum. It can lower the cost of major foundation investigations by reducing the number of expensive borings required. For this reason it should permit a more thorough foundation investigation to be made.

In recent years several methods have been developed for estimating the capacity of soil in place. Of these the following three devices have received the most attention: the rotating vane or Swedish auger, the Dutch cone penetrometer, and the standard penetration test.

The rotating vane is a device consisting of four rectangular wings mounted at 90-degree intervals about a shaft. In order to indicate the shearing resistance of the soil it is forced into the ground to the depth at which measurements are to be made. It is then twisted slowly until the soil is sheared. Since the twisting moment on the shaft required to cause this failur is equal to the moment supplied by the resistance of the soil along the failure surface of revolution, the shear strength of the soil can be calculated.

The vane was originally developed in England toward the end of the last war to provide means for indicating the capacity of soils to support tracked vehicles. It was also used by Swedish engineers for evaluating the capacity of foundations.

The cone penetrometer is essentially a small footing that is forced into the ground until the resistance of the so l is overcome. It consists of a 60-degree cone approximately 1½ inches in diameter at the base. It is connected by a thin steel shaft to an oil cylinder fitted with a gauge calibrated in soil bearing pressure units. The load transmitted through the cylinder to the cone is measured on the gauge. Soil friction on the steel shaft is preve ted by placing it inside a steel casing. This apparatus is widely used in the low countries of Europe to locate strata of suitable earing for supporting piles.

The standard penetration test is an empirical method for evaluating the capacity of foundations on sand. The capacity of the sand is measured by driving a split-tube sampler, 30 inches long and 2 inches in outside diameter, 12 inches into the soil under the blow of a 140-pound hammer freely f lling a height of 30 inches.

Initial calibrations of this test with field loading tests permitted a chart of hammer blows versus safe bearing values for granular soils to be developed. The test is not recommended for use in sensitive clays soils but there is reason to believe it may be a useful test in some of the leaner clays and glacial till soils.

The Ontario Hydro-Electric Power Commission was particularly interested in deep sounding equipment because of the large number of foundation investigations required during recent expansion activities. It was hoped that the equipment could be used to evaluate foundation conditions for transmission tower footings. As a rule, work schedules for tower line construction do not permit thorough foundation analyses to be made. Occasionally this ignorance of soil conditions results in costly delays. It was felt that a solution to the problem might be supplied by vane or cone tests.

With this in mind a program was initiated to study the accuracy and limitations of the vane and the cone. Several field measurements were made with these devices and the results compared with the laboratory unconfined compression test. The cone penetrometer used was the relatively flimsy model shown in Fig. 1. Experience with it was to govern the decision regarding purchase of a more rugged model. A limited number of tests in clay soil were made with this device and after each test an undisturbed sample of soil was obtained. Each sample was subjected to the unconfined compression test, following which the ultimate bearing capacity of the soil was calculated using Terzaghi's formula. A comparison between the two methods for measuring soil capacity is shown in Fig. 3. It will be observed that most of the points lie above the theoretical capacity indicated by soil mechanics theory. This is not considered a fault because theoretical formulae have often been suspected of underestimating soil capacity. Although there is some scattering of individual points on the chart it is by no means extreme when one considers the factors that an influence measurements of soil properties. Unfortunately the apparatus purchased was not robust enough for the soils encountered and plans are under way to obtain a sturdier rig. This comparison was made for clay soils only. A sturdier rig will permit tests to be made on granular soils.

Enthusiasm for subsoil measuring devices centred around the vane apparatus. This interest prompted the incorporation of several modifications in the machine, some of which are of questionable merit. The principle of operation is illustrated in Fig. 2. Stress is applied to the soil by rotating a 4-bladed vane 2 3/8 inches in diameter and 10 inches long. The rate of rotation must be very slow to avoid generating a temporarily high soil resistance. In passing, it is understood that a height-diameter ratio of 2 is preferred for a vane. The vane is connected by means of standard drill rods to a calibrated flat spring equipped with handles. Torque applied through the spring to the vane causes the rider on each scale to move and indicate load. Each scale has been calibrated to indicate shearing resistance of the soil directly. This was done on the basis of the formula:

$$S = \frac{M\ (max)}{\left(\frac{\pi D^2 H}{2} + \frac{\pi D^3}{6}\right)} \text{ pounds per square foot}$$

where M (max) = Maximum turning moment applied to vane in foot pounds.
D = diameter of vane in feet,
H = height of vane in feet.

The calibration assumes that the resisting force of the soil acts along the surface of revolution of the vane at a moment arm o D/2 from the centre of the shaft. Subsequent tests have cast doubt on this supposition. As in the case of the Dutch cone, soil friction on the shaft is avoided by placing the drill rods inside a 3-inch casing.

Figure 4 shows a comparison of the shearing resistance of several soils determined by the vane and by the unconfined compression test. Most of the tests were made at a depth of 3 feet below the surface. This was done to avoid any possible influence of depth on the soil strength. Experience with normally loaded clays suggests that their strength should increase with depth. Although the vane measures this increase, the laboratory test frequently does not register it.

If both tests supplied accurate measurements of the shear strength of the soil all points should lie along the 45-degree line shown. Since the laboratory test is frequently suspected of underestimating the strength of soils one might expect the average of the points to lie somewhat above the 45-degree line. This condition does exist but the extreme scattering of the data does not permit a relationship between the two tests to be established.

There are possibly two explanations for this poor agreement. One is that the samples for the unconfined compression test were so disturbed that they greatly underestimated the strength of the soil. Some disturbance during sampling is inevitable but it is unlikely to influence the results to this degree. The Commission is aware of the conditions that can produce poor samples and reasonable care is exercised to avoid them. The second explanation seems to be more reasonable. This is that the centre of the resisting forces in the soil is not located along the surface of revolution of the vane but at some other position farther out from the centre of the shaft. If this is so, a given resisting moment to the rotation of the vane can be generated by a smaller shearing resistance in the soil. In other words a longer lever arm is available to the resisting forces.

Values for the shear strengths were supplied from the unconfined compression test on samples taken 4 feet below the surface. This method of calibration is not very satisfactory if the soil properties are different at greater depths.

Figure 5 shows a conception of soil failure around a vane which might offer a method of interpreting the results of the vane test. This method assumes that the soil fails along logarithmic spirals having the equation

$$r = r_o e^{\theta \tan \phi}$$

Where r is the radius from the centre of the spiral to any point on the curve;

r_o is the radius from the centre of the spiral to the tip of the vane;

θ is the angle in radians through which the arc moves from the tip of the vane to any point on the arc; and

ϕ is the angle of internal friction of the soil.

The centre of the spiral is located on a straight line making an angle ϕ with the vane blade. This permits the arc to leave the vane blade at right angles.

Resistance to rotation of the vane is supplied by the cohesion of the soil acting along the arc of failure and the friction force F having a line of action through the centre of the spiral. Some approximate formulae were developed on the basis of this conception of failure which theoretically permit the use of the vane in both granular and cohesive soils.

For purely cohesive soils this spiral becomes a circle having a diameter equal to the diameter of the vane. Force F then passes through the centre of the shaft and applies no resisting moment. In Fig. 4 which illustrates the comparison between the vane and unconfined compression test data, good agreement can be observed for medium and soft saturated clays having a shear strength up to 1000 pounds per square foot. The angle of internal friction of these clays was probably close to zero. Unfortunately circumstances did not permit triaxial tests to be made so the presence or absence of internal friction in the stiffer clays could not be confirmed.

Whatever may be the cause of the unsatisfactory demonstration of the vane, its value to the Commission as a simple tool for measuring soil strength is considerably reduced. This is because it appears to require considerable correlation with other data before it can be interpreted. However, it is felt that it has its place in large foundation investigations where the soil profile and soil properties have been satisfactorily established. Here it should reduce investigation costs and provide more knowledge per dollar spent by disclosing possible differences in soil properties between borings. There is also reason to believe that it is an easy method for measuring the sensitivity of soils and for indicating the stress-strain characteristics of the soil prior to failure.

In the search for a simple method for obtaining an approximate measure of the capacity of the soil the Commission turned to the standard penetration test. Despite the fact that the test was primarily designed for granular soils, it has been noted during many foundation investigations that the resistance to penetration of sampling equipment is a guide to the condition of all types of subsoil.

During the last 4 years well over a dozen major and minor foundation investigations have been conducted in the process of which upwards of 2000 penetration and unconfined compression tests on clay soils alone were made. In all of these tests the penetration resistance to the sampling tube followed the same trend with depth as the unconfined compression test. This held for 2-inch and 2 5/8-inch thin-walled Shelby tubes as well as with the thicker-walled split-tube samplers. Out of curiosity a plot was prepared of the unconfined compression test data versus the penetration resistance to see if any trend could be established. The results of this compilation of data are indicated in Fig. 6. The dark blotches on the chart represent points which were too densely packed to be clearly distinguished.

Although the relationship between the two tests is by no means well marked it is not altogether unsatisfactory when one considers the wide range of soils tested and the disappointments that beset soil measurements. The limits of the denser part of the data have been defined by two dashed lines. It is felt that many of the points above the upper dashed line represent tests on soils with low cohesive resistance and a high percentage of silt and coarser soil particles. Points below the lower dashed line might well represent samples that have lost moisture before the unconfined test was made or conditions where wash water penetrated ahead of the boring before the sampler was driven.

Although the Commission is fully aware that this data should not be used for the final design of important foundations, it is felt that measurements of driving energy are an excellent guide in the appraisal of the capacity of soils. This is especially so in the case of transmission tower footings where simple, approximate test methods can be tolerated. With this in mind an arbitrary line was drawn through the most dense concentration of points. This line represents the approximate relationship between the penetration test data and the laboratory unconfined compression test. The line toward the upper dashed boundary line was purposely plotted in order to insert a small factor of safety into the estimate.

Since the ultimate bearing capacity of soils supporting square and continuous footings is approximately 3.7 and 2.85 times the unconfined compression strength respectively, this chart can be readily converted to indicate bearing capacity.

By applying a factor of safety of 3 a relationship of safe bearing capacity versus driving resistance in clay soils can be established. This is illustrated in Fig. 7. For comparative purposes this chart also contains safe bearing values for footings on sand interpreted from data in "Soil Mechanics in Engineering Practice" by Terzaghi and Peck.

In closing, what are considered to be the merits of the modified penetration test are summarized.

(1) The test is as easy to carry out as other deep sounding methods in glacial soils;

(2) it is a useful guide to the capacity of lean clays and other cohesive soils of glacial origin;

(3) it permits the recovery of soil samples while a sounding is being made;

(4) it serves the same purpose as any other sounding device for probing for variations in soil conditions over a site; and

(5) it should be suitable for minor footing designs such as for tower footings where the time and expense of more complete analyses cannot be spared.

Discussion

The chairman stated that during the war, the Associate Committee on Soil and Snow Mechanics had investigated penetrometers for a particular purpose and had found that the state of stress in soil underneath a sampler was a very complex one.

Mr. Lea mentioned that his Company had developed a vane apparatus which recorded a stress-strain diagram. He said that in his experience, the vane test gave approximately twice the value of shear strength obtained by the unconfined compression test but this ratio varied with soil types. He thought that there was a great need for correlation of field penetration tests and of the results obtained from them. He said that he had studied different types of soil samplers and had found the results of tests on Shelby tube samples hard to interpret to obtain an indication of the cohesive strength of soil.

Referring to Fig. 6 in Mr. Trow's paper, Dean Hardy thought that different curves would be obtained for each type of soil, often varying widely. He said that one soil type should be dealt with alone and then other soils could be correlated against the one standard result. Then a series of curves would be obtained each of which would apply to a certain soil type.

Dean Hardy pointed out that if the standard penetration test were used that a sample should always be obtained. In this way, one curve could be established for sand, one for silty materials, and so on.

Mr. Trow said that the figures he had given were just to enable field engineers to make an on-the-spot decision and that they were never intended for accurate analyses. Mr. Trow said that Dr. Terzaghi did not recommend the penetration test in sensitive clays, but as most clays encountered in Hydro work in Ontario were lean clays, the results were thought to be satisfactory.

In discussion of Mr. Trow's talk, Mr. Fairbairn stated that Construction Borings Limited had used the vane borer for some years as a check on unconfined compression test and standard penetration test results. Results were generally good in soft and cohesive materials where the vane test usually confirmed the unconfined compression results, whereas the standard percussion test indicated lower bearing values and shear strengths in these materials. This, of course, was quite apart from the question of settlement caused by consolidation of the soil. Mr. Fairbairn said that the Company used the standard penetration test with the split spoon sampler in coarse sands and gravels, and switched to vane tests and Shelby tube samplers in cohesive materials. He called the attention of those present to a reference (1), concerning the vane borer where numerous experiences with the apparatus were reported, along with comparative tests with cone penetrometers and unconfined compression tests.

Mr. Torchinsky asked how the vane sampler was pushed into the soil and Mr. Fairbairn replied that it depended on the type of soil. Usually it could be pushed by hand, but sometimes it was necessary to drive the vane with a hammer.

Mr. Schriever asked if the shearing resistance along the bottom plane of the vane contributed much to the total shearing resistance. Mr. Trow said that he thought no one could really say where the shear failure occurs. He thought the soil attained its greatest strength just before failure and that the potential sources of soil resistance were outside the diameter of the vane blades. In soft clay, the concept of failure along the circumference of the circle made by the vane is correct but if the soil possesses internal friction, then this must be taken into account. Mr. Trow thought that more study should be given to the exact location of the plane along which resistance was developed. In any case, he thought that the vane should only be used when the soil properties were generally known, and when other tests were made for correlation purposes. Mr. Fairbairn was in full agreement with this.

Mr. Lea remarked that as failure in stiff clay progressed, the stress was reduced, but in plastic materials, this was not the case.

In reply to a question asking if the shear decreased in relation to the rate of application of stress, Mr. Trow replied that in tests done by Swedish engineers, it had been found that the vane could be turned through one degree per minute. Mr. Lea thought that the total length of time for the test was also an important consideration and varied with the material. He suggested that the total length of the test should be 5 minutes to tie in with conditions in the unconfined compression test. The time of test would vary, however, with each type of material tested. The residual shear strength was determined by completing two revolutions of the vane and then reading the remaining shear resistance.

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(1) adling, L. and S. Odenstad. The van borer - an apparatus for determining the shear strength of clay soils directly in the ground. Royal Swedish Geotechnical Institute Proceedings No. 2. S ockholm, 1950.

Dean Hardy said that if the load was applied too quickly in vane tests then the value obtained would be too high. In silty materials, sometimes a reduced value would result. Both Mr. Trow and Mr. Lea said that they had not used the vane test for silty soils but only in clays.

Mr. Baracos said that since the results of the vane tests were directional, correlation should be attempted with unconfined compression tests cut horizontally and vertically. He asked if any correlation had been made with stratified clays. Mr. Lea said that he had used the vane test in varved clays near Sault Ste. Marie where a natural slide had occurred. He said that correlations had been good with the strength of the unconfined compression test being about one-half that obtained with the vane test. Mr. Baracos felt that in general, vane tests might be too directional and hence unsuitable for laminated clays.

Mr. Ripley asked if anyone knew of correlation tests which had been run using sample spoons of various sizes and end areas, driven with various energies. He considered that such tests would be valuable. As far as could be determined, no such tests had yet been done.

Mr. Trow said that a 300-pound hammer with a 12-inch drop was used in Hydro work. Tests were done with 2- and 2½-inch and split-tube samplers and results were very scattered. No adjustment was therefore made for end area in their tests, since no significant difference was found with different types of samplers of the same size, whether or not a sample of soil was present in the tube.

Mr. Peterson remarked that in many penetration tests a drilling rig with a 140-pound weight was used but in hand auger work this size of weight was very cumbersome. He wondered if anyone knew of any simple method to remedy this situation. Mr. Lea said that he had tried to establish correlation between the 140-, 300-, and 500-pound hammers and that reasonably good results had been obtained in firm materials as far as energy was concerned. In soft materials, however, the sample advanced under the weight of the hammer alone.

Mr. Fairbairn thought it would be difficult to establish correlation, especially in sand. He said that in some cases for 11 blows the sample would penetrate 11 inches but on the twelfth blow it might advance a foot.

Mr. Peckover remarked that this was the second Conference at which this problem of correlation of penetration resistance of various sizes of hammers had been discussed. He suggested that if correlation of large and small weights was attempted some small standard weight be used, such as 30 pounds. Mr. Torchinsky thought that the suggestion was a good one and that it would be desirable to have standard weights and procedures for penetration tests. In this way, the results of any tests would be valuable elsewhere.

Fig. 1

The Dutch Cone Penetrometer

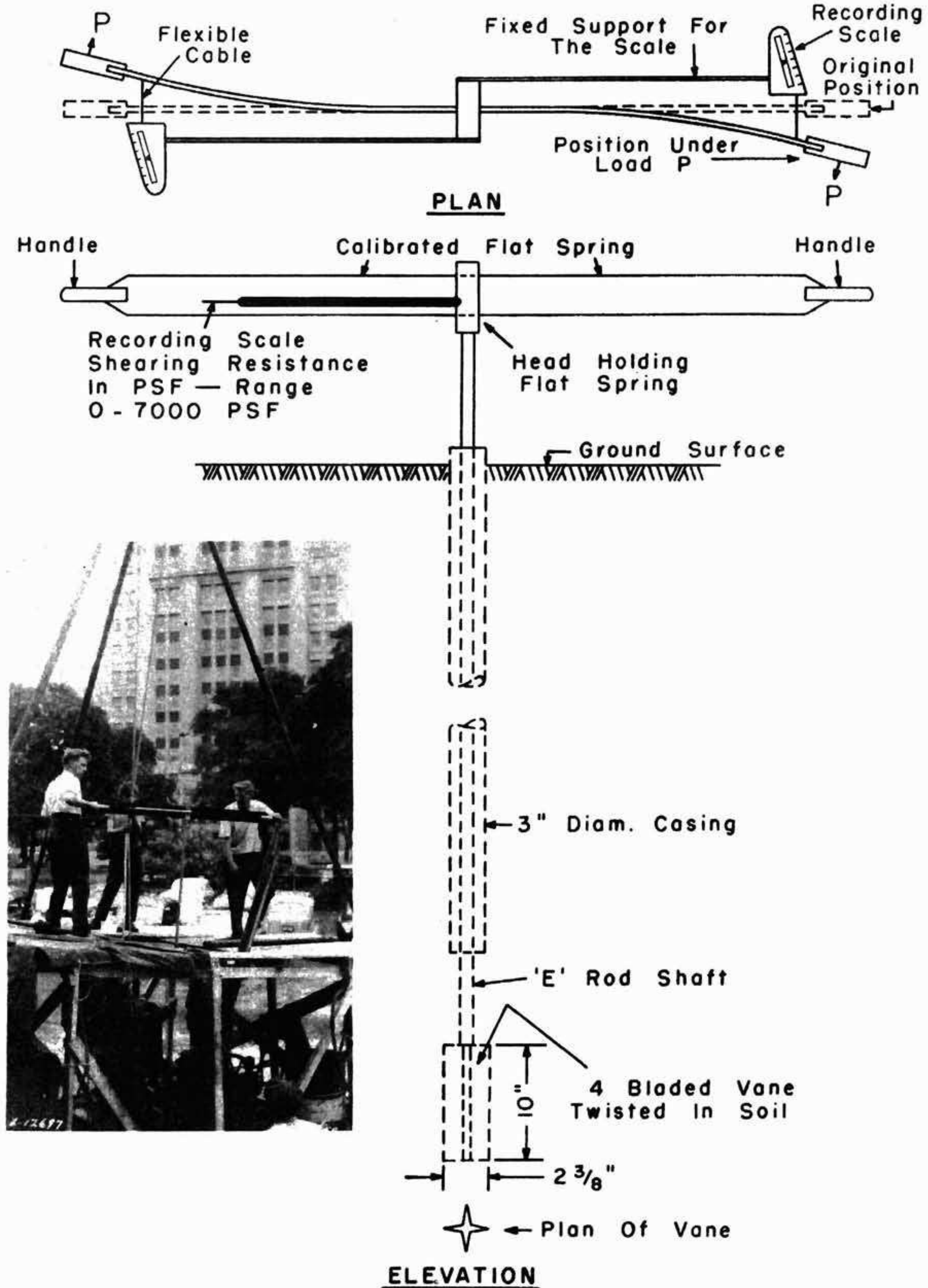


FIG. 2 - SKETCH ILLUSTRATING THE PRINCIPLE OF THE VANE TEST

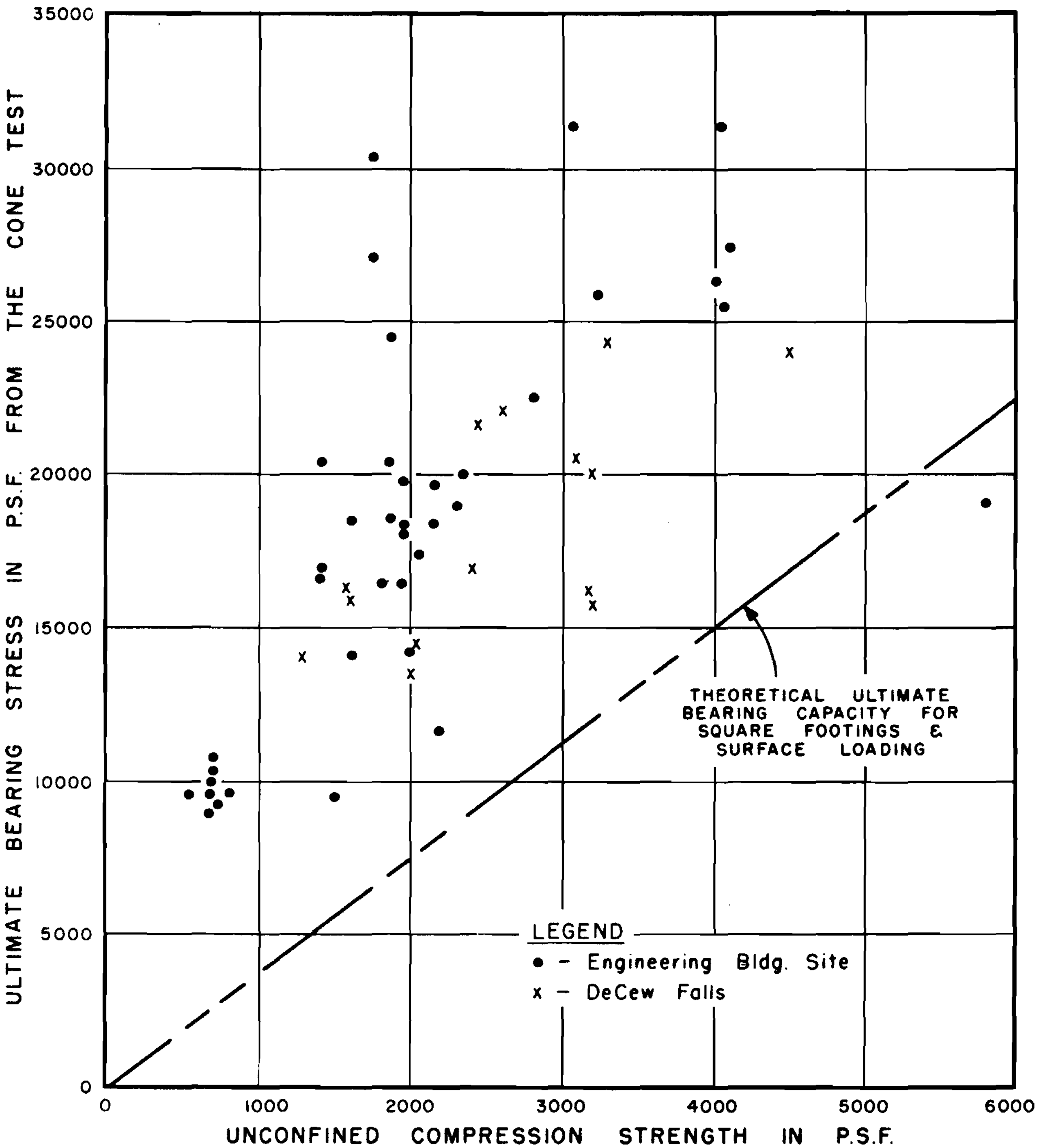


FIG. 3 - COMPARISON OF CONE TEST RESULTS WITH DATA FROM THE UNCONFINED COMPRESSION TEST. THE THEORETICAL CAPACITY LINE IS OBTAINED FROM SEMI - EMPIRICAL EQUATIONS DEVELOPED BY K. TERZAGHI.

DWG. NO. 14972 - R.D. - OCT. 12/50 - T.W.

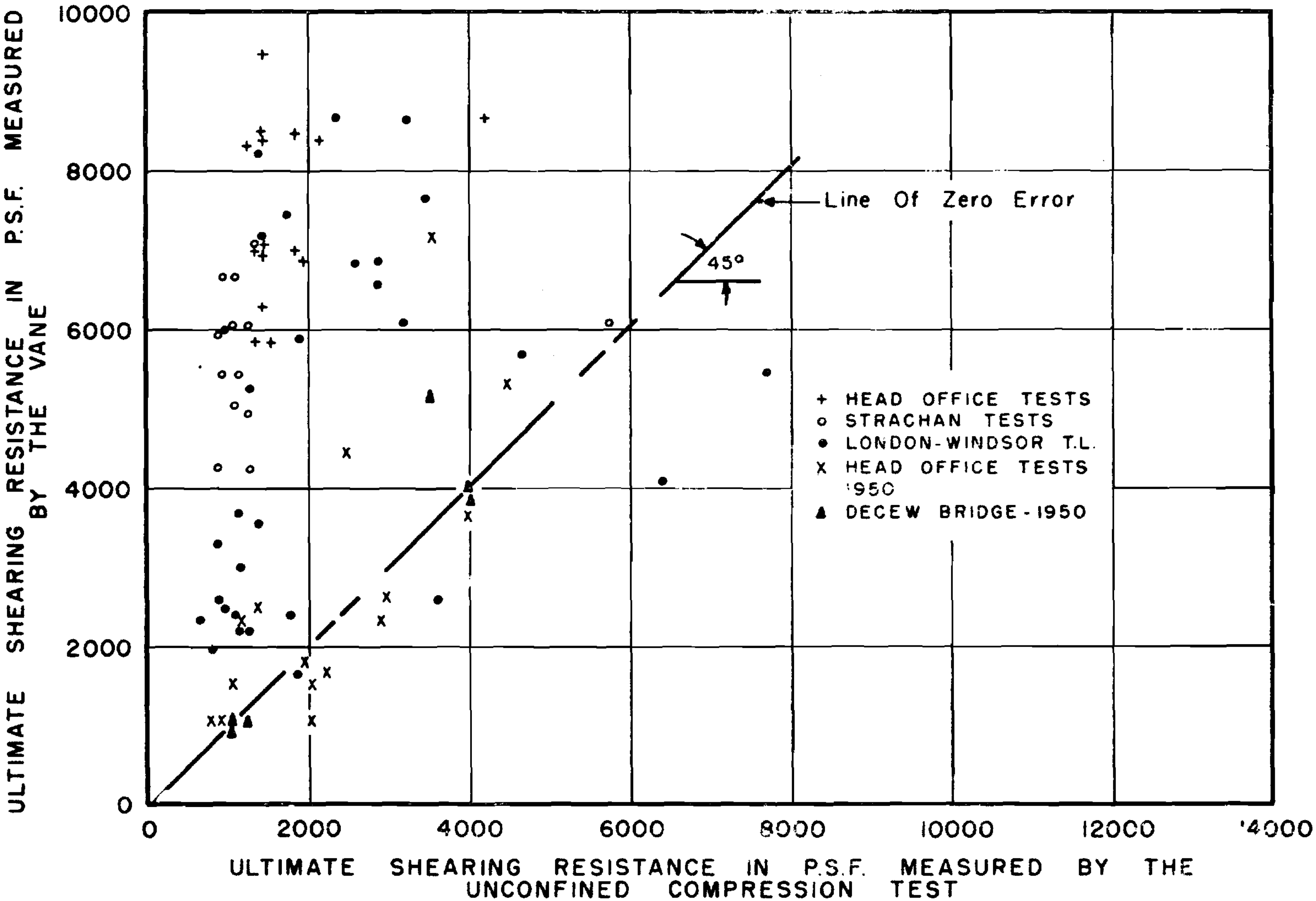


FIG. 4 - CORRELATION BETWEEN THE SHEARING RESISTANCE MEASURED BY THE ROTATING VANE & THE UNCONFINED COMPRESSION TEST

DWG. NO. 14973 - R.D. - OCT. 11/50 - T.W.

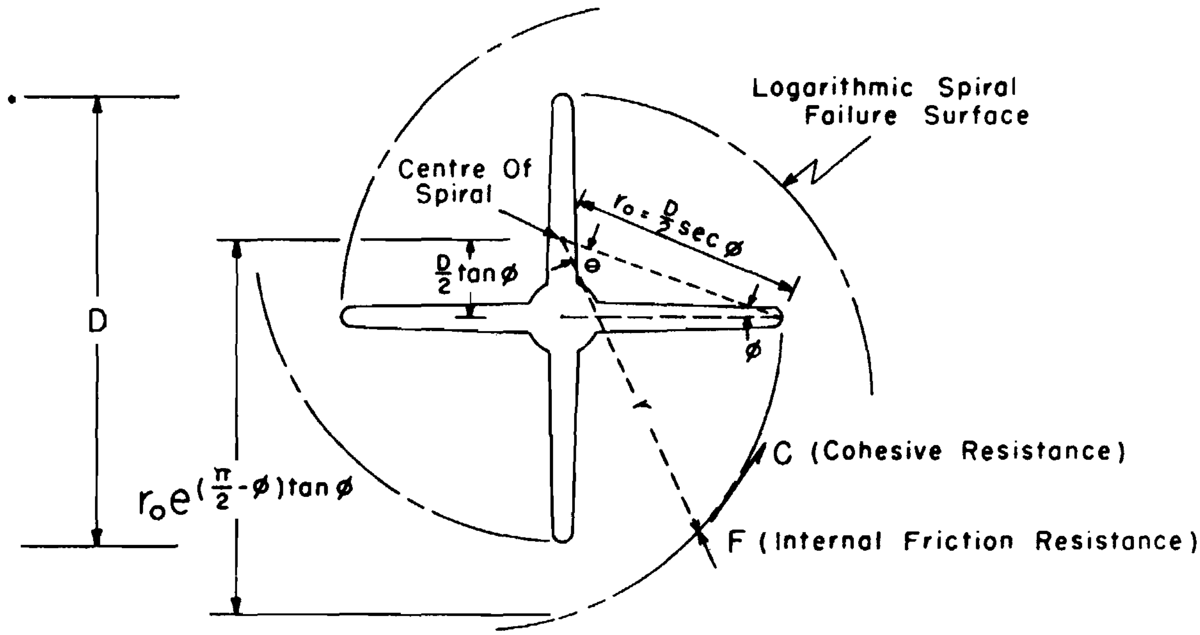


FIG. 5 - SKETCH ILLUSTRATING POSSIBLE POTENTIAL SURFACES OF FAILURE ALONG WHICH RESISTANCE TO ROTATION OF THE VANE IS DEVELOPED

DWG NO. 15553 - RD - SEPT. 17/51 - T.W.

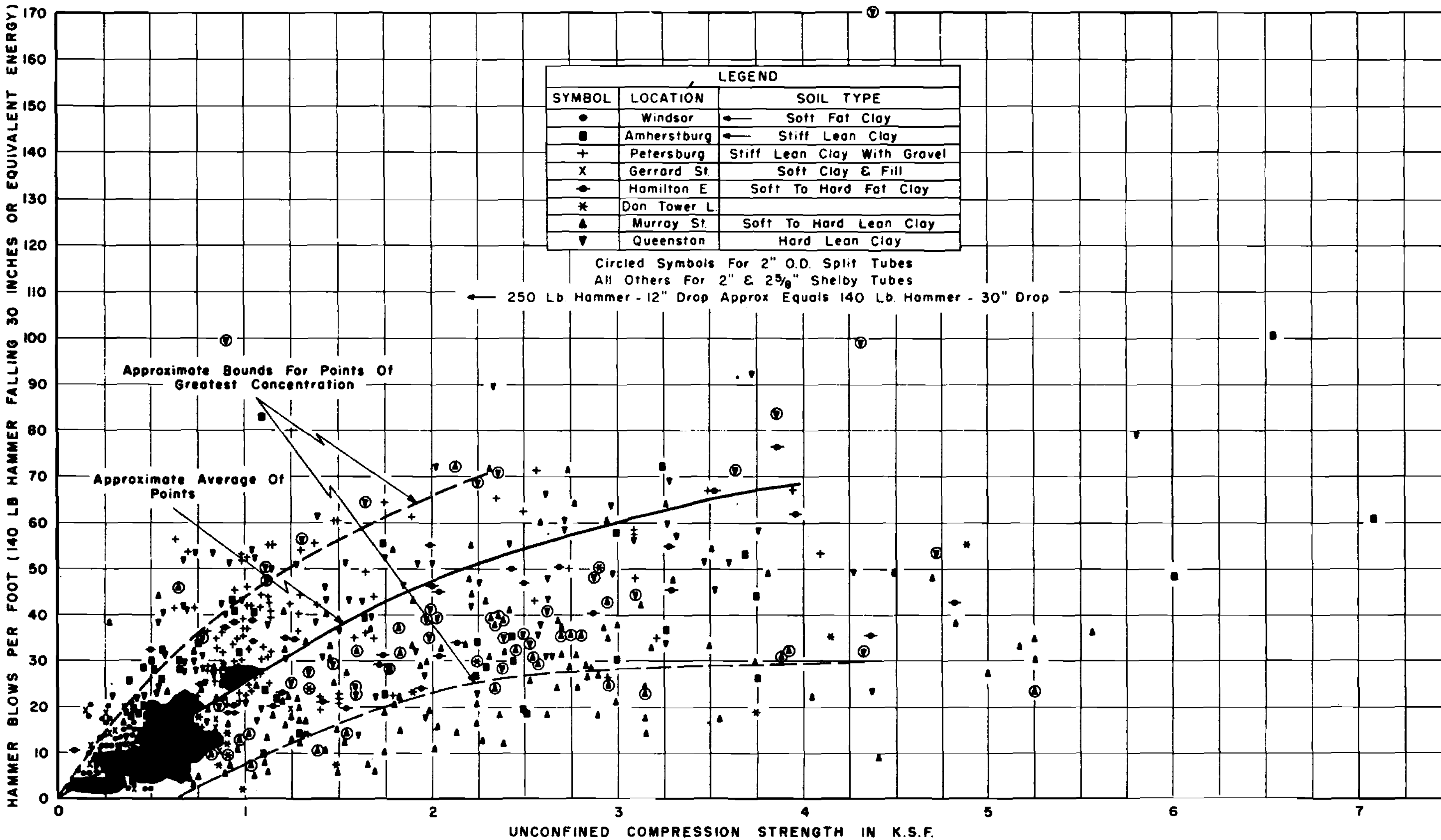


FIG. 6 - COMPARISON OF DATA FROM STANDARD PENETRATION TEST & UNCONFINED COMPRESSION TEST USING SPLIT & SHELBY TUBE SAMPLERS IN CLAY SOILS

DWG. NO. 2615 - RD - SEPT. 19/51 - T.W.

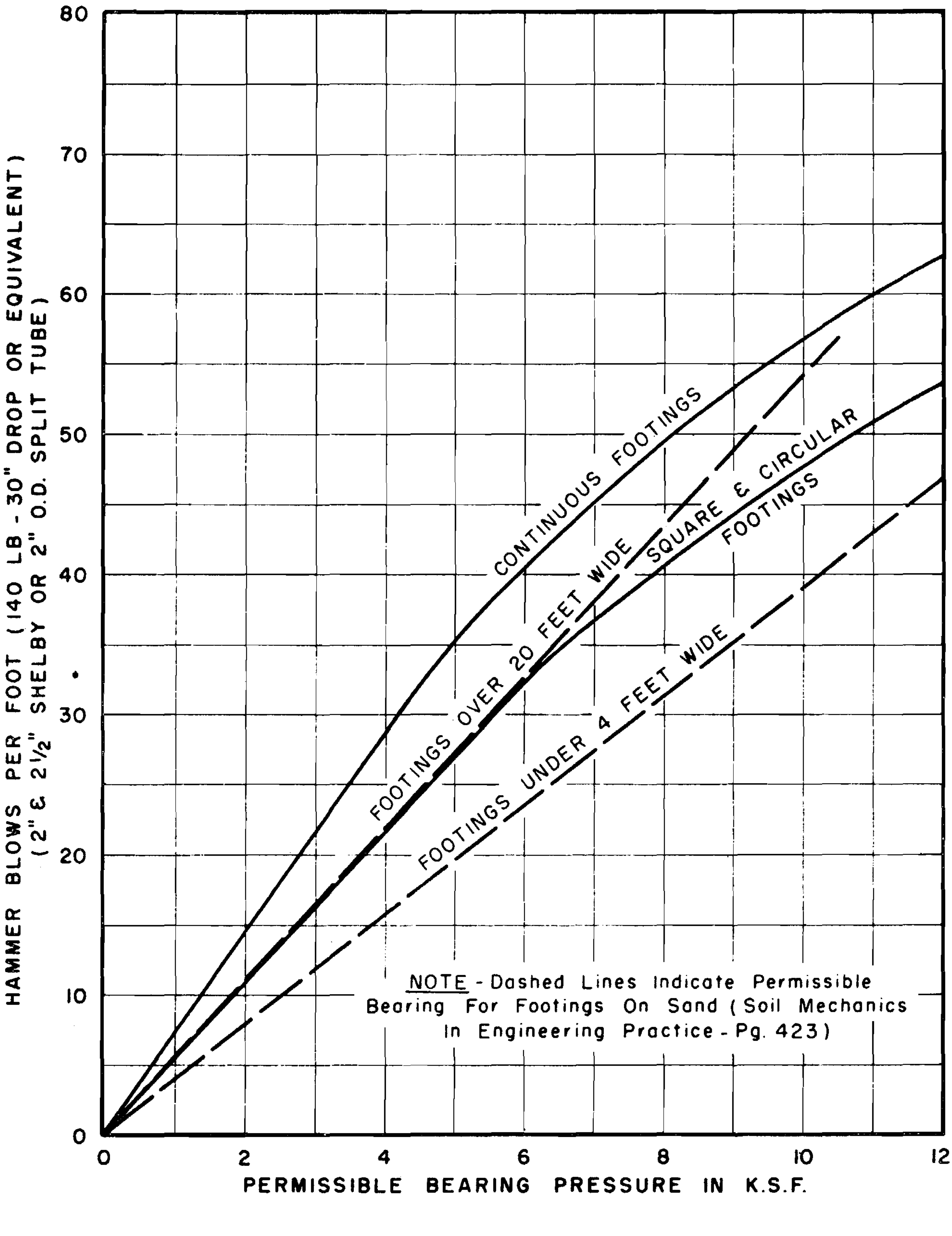


FIG. 7 - EMPIRICAL METHOD FOR ESTIMATING THE SAFE BEARING VALUE OF SOILS ON THE BASIS OF THE MODIFIED STANDARD PENETRATION TEST

DWG. NO. 15569 - RD - SEPT. 19/51 - T.W.

Section 5

The Swedish Steel Foil Sampler

by

N. D. Lea

About the year 1800, boring and sampling equipment was developed for the exploration of coal deposits. This equipment was widely used for coal exploration in the early part of the 19th century. In the latter part of the century the same methods were adapted, almost without modification, for civil engineering purposes. But it was not until early in the 20th century that they were used at all extensively for civil engineering work.

During the first half of the 20th century there have been some refinements in these procedures, but no basic changes. The same basic principle of pushing a pipe into the soil, which was used for the early exploration of coal, is commonly used today for soil investigation. It has been learned that thinner pipe gives better samples and that putting a piston inside the pipe improves the situation even further. A few tricks have been developed for holding samples under special conditions. It must be recognized, however, that there are some basic shortcomings to this method of soil investigation for civil engineering purposes.

The standard intermittent sampling procedure is to take a sample every 5 feet. To study a 50-foot deep deposit requires 10 sampling operations. Using Shelby tubes 2½ feet long, this provides samples of half of the strata. Even if a good recovery is obtained in each sample, which is not always the case, it is known that the material at either end of the sample is disturbed to such an extent that it is not satisfactory for laboratory tests. Certainly, no more than the middle 18 inches would be reliable for laboratory work. This means that actually only 15 feet of the 50-foot strata has been sampled in a way to give good samples for laboratory testing. If a more intensive study of the soil is required, then the sampling might be done continuously in the same bore hole. By this procedure, however, at least a few inches of material is lost between each sample because the bore hole must be washed out. Therefore, with 20 operations, the best possible recovery of good core or sample would be less than 30 feet. There is still a good possibility that the most important information--which in the case of the study of the stability of slopes would be sand layers or softer layers of clay--might be lost at the space between samples or by dropping out of the bottom of a tube. To improve the information still further with present equipment it is necessary to make two borings, side by side. Using two such borings with 40 sampling operations, it is possible with present procedures to obtain a continuous core of the soil. The only problem is to relate the samples one to another for, with the present equipment, it is not possible to tell accurately the depth from which a sample has come. With good luck the samples can be correlated by their stratification.

Some engineers have been working for some time on methods which are particularly adapted to the civil engineering requirements for soil sampling. In 1943, Swedish engineers of the Royal Swedish Geotechnical Institute began working on the idea which is illustrated in Fig. 1. A piston is held stationary inside a tube by means of a chain extending to the surface. To the piston are fastened a number of thin steel foils which lie between the soil and the tube, each in a vertical plane in such a way that they completely isolate the soil from the tube and thereby eliminate friction. The lower end of these foils go through slots in the sampler head and then up into magazines where the supply of foils is stored. After several trial models of this sampler had been built and tried out in the laboratory and the field, Model 5, which is illustrated in Fig. 2, was constructed. This model benefited from what had been learned in the earlier models of the requirements of rigidity for field operation and of careful design to obtain good samples. With this sampler head it is now possible to obtain continuous cores of soil up to 130 feet long. The limitation of the length is only in the amount of foil that can be stored in the magazines and in the strength of the foil to carry the soil. This means that now, instead of requiring 40 sampling operations to get a continuous sample of good core from a 50-foot deposit, only one continuous operation is required.

The sample is removed from the ground in sections of tube about 8 feet long. The steel foils are still in place between the soil and the tube. The sample is extracted in the laboratory by simply pulling on the foils. The force required is usually less than [illegible] pounds. The diameter of the core, 2 3/4 inches, has been designed to fit laboratory equipment.

The details of the sampler head and its operation will not be given here for they are described completely in the Proceedings No. 1 of the Royal Swedish Geotechnical Institute, 1950.

There are some obvious limitations to the use of this equipment. It cannot penetrate rock. Neither can it, in its present form, penetrate very hard sediments or deposits which contain boulders and coarse gravel. The sampler in its simplest form with manual above-ground equipment is suited only for sampling clay deposits with an unconfined compressive strength up to about 3 tons per square foot. Jetting equipment, illustrated in Fig. 3, and rotary drilling equipment, as illustrated in Fig. 4, have been used successfully, however, for taking continuous cores of sands and of stiffer and denser materials than can be sampled by pushing alone. Development work is still proceeding on this equipment.

There are a number of interesting advantages to this equipment over the intermittent sampling procedure. In the first place, the sample is continuous and this of course is the greatest advantage. The sample, however, is also in a much better condition than those obtained by Shelby tube and piston type samplers.

Furthermore, the equipment is much easier to operate in cold weather than the older equipment because no water is used in the operation. The heaviest piece of the manual equipment can be carried by two men. Finally, for shallow depths the cost of the operation is found to be about the same or perhaps slightly less than intermittent sampling every 5 feet. For greater depths, the advantage in cost of the new equipment becomes much more pronounced, for the costs do not increase but rather decrease with depth. There is no more work required to push the sampler from 100 feet to 110 feet than there is in pushing it from 10 feet to 20 feet. On the other hand the cost of intermittent sampling increases tremendously with depth. Therefore, under suitable conditions the new sampler gives a continuous sample at less expense than that required to produce inferior intermittent samples with older equipment.

Canadian patents covering the steel foil sampling equipment are held by the Swedish inventors. This equipment is operated in Canada, however, by the Foundation Company of Canada Limited under agreement with the inventors. Arrangements will be made to have a sufficient number of rigs, with skilled operators, available to satisfy the requirements of all who wish to use the equipment in Canada. At present only one manually operated rig is in Canada and this is stationed in Montreal. It has already been used as far west as Winnipeg, however, for its shipping weight is only one ton. It is anticipated, however, that as other regions have sufficient demand for the equipment that rigs will be stationed in other parts of the country. It is also anticipated that when there is seen to be sufficient use for it, a machine-operated rig equipped for jetting and rotary drilling will be set up.

Demonstration of Apparatus

Following the luncheon recess, delegates of the Conference gathered in the field to observe a demonstration of soil sampling with the Swedish steel foil sampler. Equipment and labour for the demonstration were arranged by the Foundation Company of Canada.

The apparatus used was the manually operated type described by Mr. Lea. During the morning the sampler head had been pushed to a depth of 25 feet in ground consisting of 11 feet of very stiff brown clay underlain by firm plastic blue clay. During the demonstration the sampler head was pushed down to a depth of 32 feet and then the withdrawal procedure begun. Mr. Lea described the various steps of the operation.

On the second day of the Conference, delegates visited the Soil Mechanics Laboratory of the Division of Building Research and examined the soil core which had been obtained by the sampler. The technique of removing the core from the sample casing was demonstrated. The core itself was in 4 lengths, each 8 feet long, representing the complete soil profile of the borehole from which it was taken.

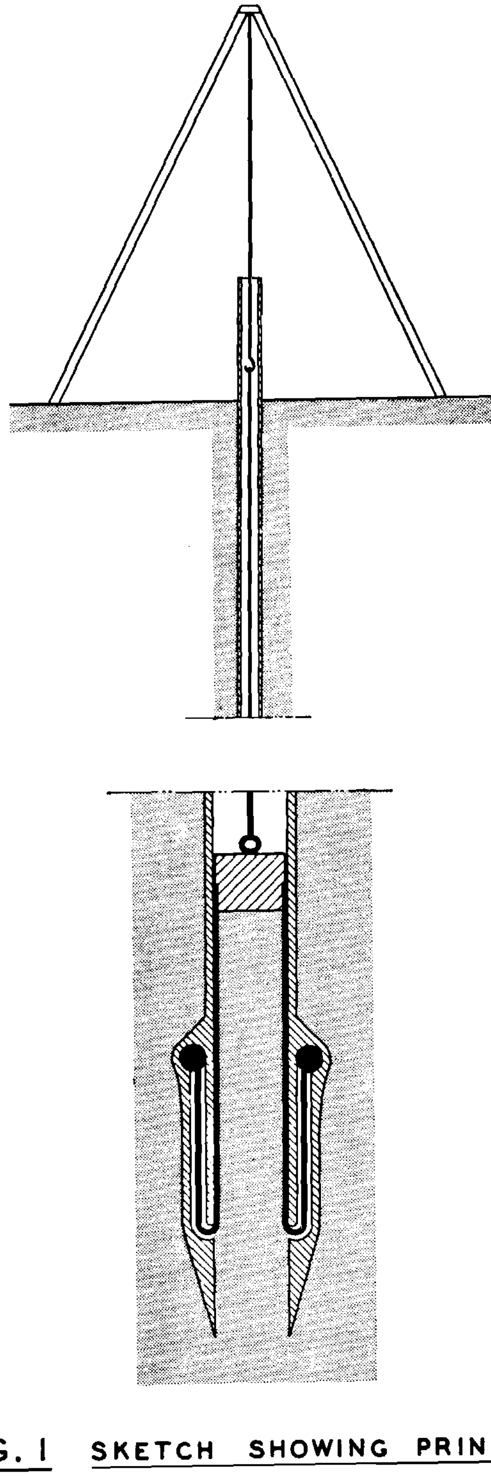
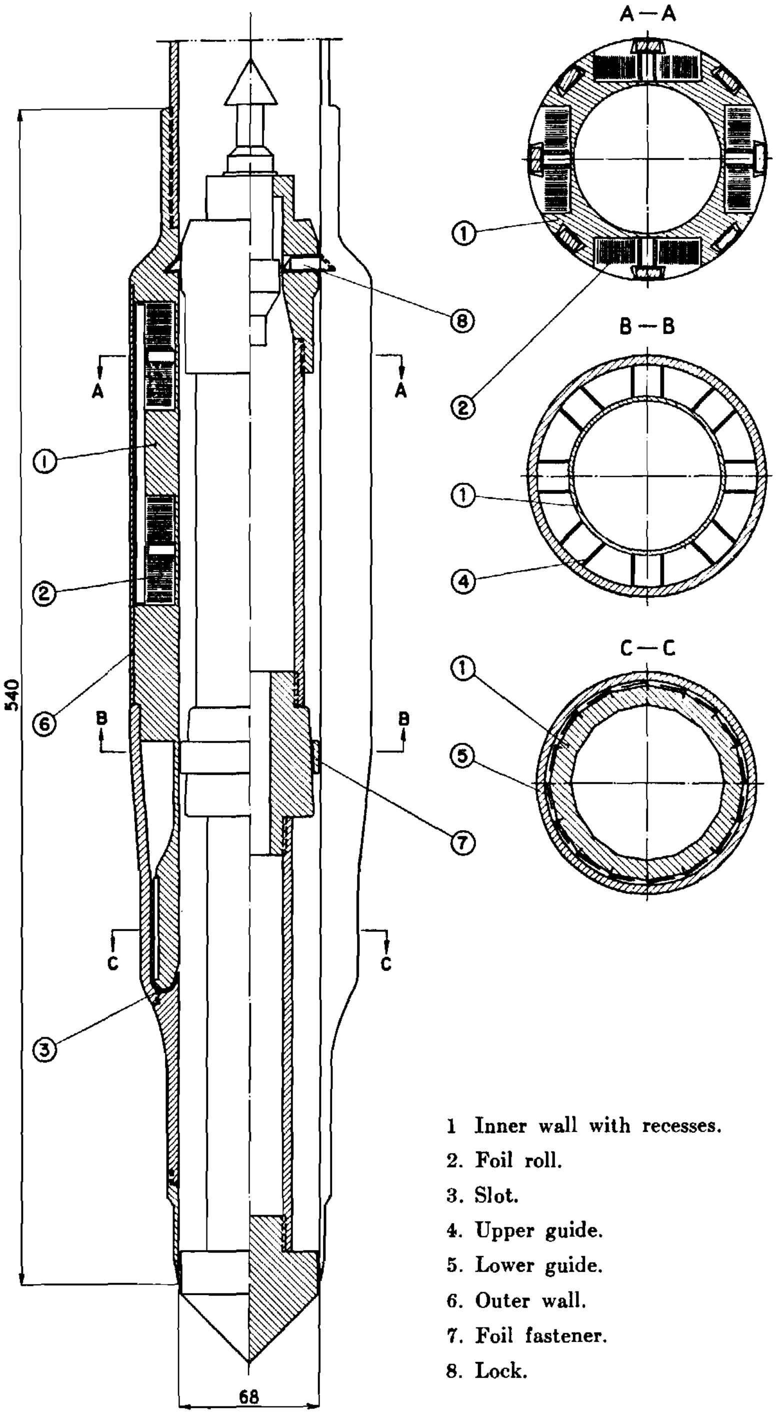


FIG. I SKETCH SHOWING PRINCIPLE OF NEW SAMPLER.



1 Inner wall with recesses.
2. Foil roll.
3. Slot.
4. Upper guide.
5. Lower guide.
6. Outer wall.
7. Foil fastener.
8. Lock.

FIG. 2 SAMPLER HEAD MODEL V.
(FOILS SHOWN ONLY IN ROLLS.)

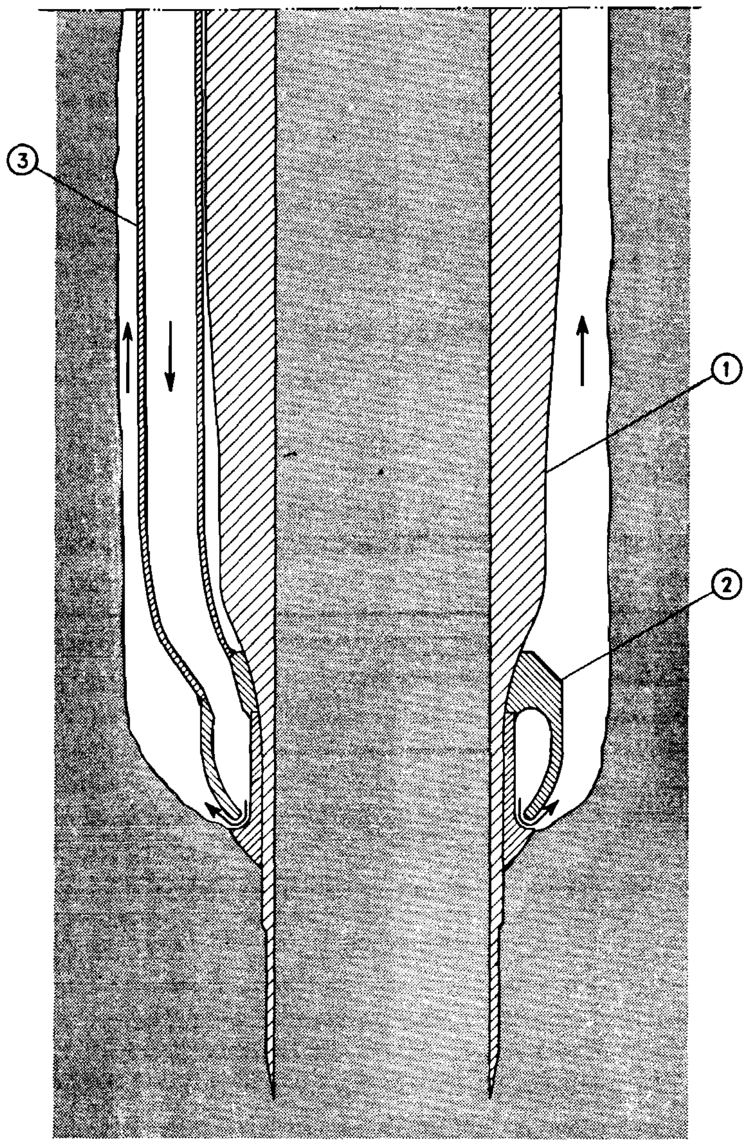


FIG. 3 SAMPLER DRIVEN BY JETTING WITH WATER.

1. SAMPLER 2. NOZZLE

3. WATER SUPPLY TUBE

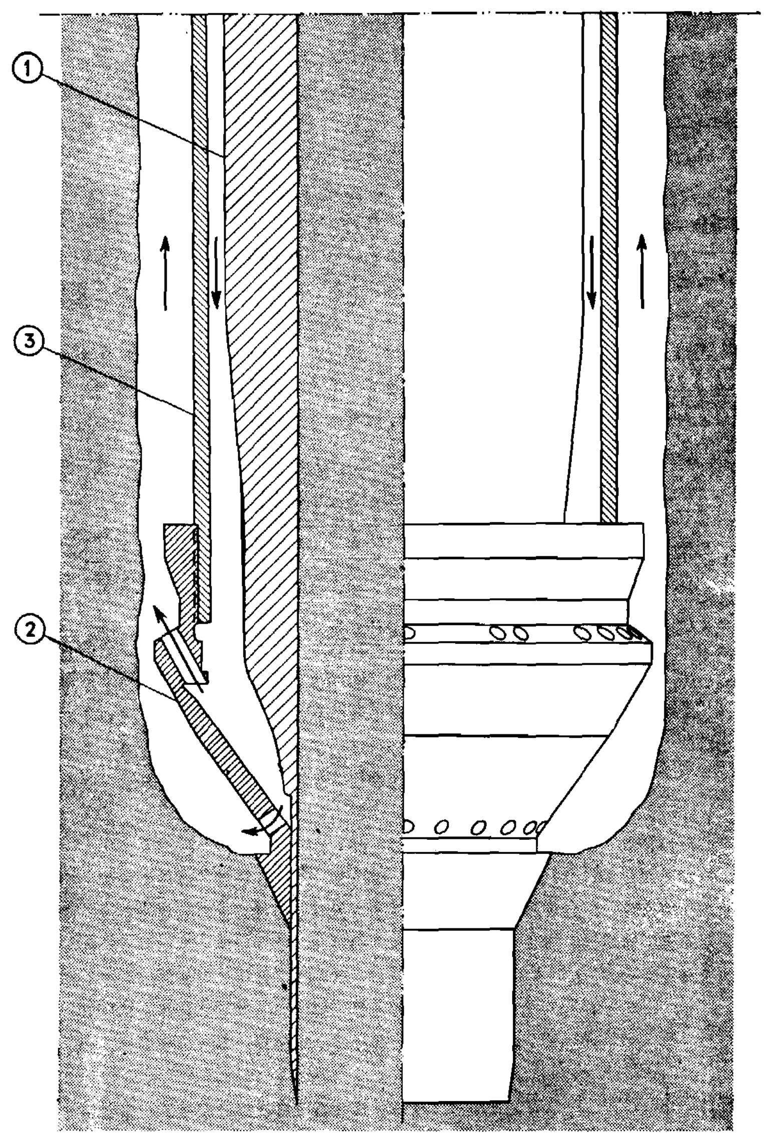


FIG. 4 SAMPLER DRIVEN BY JETTING WITH DRILLING FLUID.
1. SAMPLER 2. NOZZLE
3. OUTER TUBE

Section 6

The Reclamation of Tidal Marshlands in the Maritime Provinces of Canada

by

L. W. McCarthy

The Maritime Marshland Rehabilitation Act passed by the Government of Canada in 1948 provides in part for the construction and re-construction of works by the Government of Canada to protect the marshlands of the Maritime Provinces from tidal flooding. The agency established to carry out this provision of the Act is called the Maritime Marshland Rehabilitation Administration which functions directly under the Canadian Department of Agriculture, with headquarters in Amherst, N.S.

The bulk of the marshland with which the Maritime Marshland Rehabilitation Administration is concerned perimeters the Bay of Fundy, body of water characterized by having the greatest known tidal range in the world, varying from [illegible] feet at the mouth of the Bay to 53 feet at the upper reaches. The tidal wave progressing across the Atlantic is accelerated through the mouth of the Bay, due to the constriction, [illegible] a velocity of about 3 miles per hour. Much higher velocities are encountered elsewhere in the Bay, in some locations as much as 1[illegible] miles per hour.

The swirling waters erode a great amount of silt which is deposited as the velocity of the tide is checked in the ascent of the rivers or by overspreading the adjacent marshes at high tide. It is generally agreed that the Fundy marshes have been and are being built up in this manner rather than from materials carried down by the rivers. At the daily low tides, the level of salt water is below the bottom of the rivers and the upland run-off drains into the sea.

Along the tributaries of the Bay, the depth of silt deposit varies considerably. Some drillings have indicated depths of 130 feet and traces of only slightly decayed vegetative growth have been located at depths of 120 feet.

The diked marshlands of the Maritime Provinces have been considered vital to the agricultural economy. There is a general scarcity of fertile soil and as the situation has developed, upland farms are dependent on the marshlands to balance farm units. Communities have grown and thrived around these areas. The same protective works which keep the tide off the marshes are in many cases protecting community services as well as railroads and highways.

The basic structures required to secure the marshlands from tidal flooding are dikes and aboiteaux.

Dikes are conventionally very low earth dams of variable cross-section ranging from 1 to 10 feet in height. They are considered as salt water dams only, as in most instances no fresh water ponding takes place.

Aboiteaux are the major dam structures built to form the link in the dike structure at fresh water outflow locations. They are usually built of brush and marsh mud, vary greatly in cross-section and range in height from 10 to 30 or more feet. Sluiceways are installed in the aboiteaux to permit the passage of fresh water from the marsh. These have tide gates to prevent the inflow of salt water with the rising tide. They are integral parts of the aboiteaux structure.

The early French settlers were the first to realize the economic possibilities of the marshlands and the first to occupy them. They built many miles of dikes and many aboiteaux using shovels, spades, horses where they were available, and an unbelievable amount of hand labour.

The general policy was to dike far enough up the rivers and streams to make aboiteau installation relatively safe and sure. Once the tide had been shut off and the run-back after high tides eliminated it was possible to cross farther down these rivers and streams with a consequent reduction in dike length. This step would not be taken, of course, until the maintenance of the lengthy dike became too costly.

The accomplishments of these early settlers were tremendous. The members of the Maritime Marshland Rehabilitation Administration can realize full well what handicaps and heartbreaks were endured by them in their grim fight to shut out the relentless tides.

It is only within the last decade that any departures from the original methods of construction have been made and these have been slight. It is a major aim of the Administration to apply present-day engineering knowledge and construction techniques to the problems of dike and aboiteau building in order to provide better protective structures with longer lives and lower maintenance costs.

A thorough investigation from the soil mechanics aspect is thus necessary and considerable progress along this line has been made to date. Much of the information obtained is of a negative nature. The soils encountered are so wet and soft that no reliable figures for design can be obtained from conventional laboratory study. To further emphasize this, mechanical analyses of samples taken from various marshes show the material in most cases to be predominantly silty with small percentages of fine sand and clay. To the Maritimer this is known as marsh mud. Liquid limits will vary from 20 to 30 per cent and plastic limits from 16 to 30 per cent. The natural water content is anywhere from 20 to 100 per cent and higher. Densities vary from 80 to 120 pounds per cubic foot. The material is generally so weak that it is very difficult to determine strength values.

Had the early settlers not carried out their program of construction and had the Administration not already carried out an extensive construction program during the last two years one might easily conclude that there was no solution to the problem within economic limits. The existing works, however, prove that the structures can be built with success and also furnish full-scale laboratory models from which it is expected design facts can be obtained which will be of use not only in the work of the M.M.R.A. but in soil mechanics practice generally.

Dikes now are generally constructed by draglines using material excavated from the marsh so as to leave the borrow pits on the outside (seaside) when the dike is completed. Borrow pits are excavated so that they will drain completely at low tide which enables them to fill with silt within a few years. (It is of interest to note that the borrow pits do not silt up if they are not drained.)

While bulldozers are also used in dike construction they are practical only under the best conditions. They are extensively used, however, on aboiteau construction to spread the marsh mud over the brush mats. As these mats are from 12 to 18 inches apart vertically, little difficulty is encountered.

Most of the dikes are built of material well above the liquid limit and slipping failures are the exception rather than the rule. When slips occur, it has been found best to leave this portion of dike for a period, allowing it to partially dry out before proceeding with construction. The drying process is not a full dewatering -- the material is usually beyond the liquid limit when finally placed to line and grade.

Dike foundations often cause considerable concern. In some instances the foundation has settled considerably causing uplift in the borrow pit. While there is settlement of the foundation owing to the dewatering effect of the borrow pit, this has not been found critical as yet. Levels are taken from time to time to enable this to be checked more fully over a period. Generally it can be foretold from experience where extreme settlement is liable to occur and this can be prevented by brushing heavily prior to construction.

As yet, the maximum height of dike which can be built without causing foundation failure has not been determined but this is one of the aims of the Maritime Marshland Rehabilitation Administration.

A point of interest when considering settlement is the amount of freeboard provided. Dikes are built so that they will be 2 feet above the maximum high tide known to have occurred after the initial construction settlement has taken place (neglecting the Saxby tide). Comparison with standard practice on dam construction would tend to make one feel that the freeboard provided is insufficient. However, the following points have to be taken into consideration:

1. The tide heights for every day of the year are available within close limits at the beginning of each year. Peak water elevation is thus known with much more certainty than can be predicted for an ordinary fresh water dam;

2. There are two complete tide cycles each day, thus, even when the extreme high tides occur, they are against the dikes for only a short period and ample time is available for the repair of breaks when the tide recedes;

3. This standard of construction represents a high factor of safety when compared with previous practice.

Dikes, as built by machines, are usually finished to 2:1 or 3:1 slopes. There is enough natural cohesion to hold the material together for a few years in even some of the most severely exposed locations. In some locations this gives sufficient time to permit vegetative growth to start and this affords relatively permanent protection to the dike. Efforts are being made by other branches of the Administration to develop types of vegetation which will grow rapidly and densely on new dikes.

In the most severely exposed regions, however, something more is generally required. The cost consideration makes gravel facing and stone rip-rap prohibitive. The early settlers tried a great variety of facing types but the one which shows most promise is the one which has been developed quite recently at Grand Pré, N.S. This is a plank facing, the planks sized and installed on end, battened and fastened to the dike by walers bolted to anchors in the dike. The facing is installed almost vertically, thus giving some compaction to the marsh mud behind it. Exposure to the salt water assures a long life to this type of facing. The work of installing the facing must be carried out fastidiously to assure that it be water-tight. If the proper precautions are not taken, the resulting facing has very little value. This facing can be economically installed and has been accepted by the Maritime Marshland Rehabilitation Administration as the standard dike finish where exposure conditions demand added protection.

Before leaving the subject of protective facing it would be well to mention one major problem which makes high-cost facings unpractical; that is bank and foreshore undercutting. As channels shift, the foreshore is cut from one side and the material deposited on the other. This makes it necessary to move the dike back to prevent failure.

Much work has been done in the past in an effort to control the channels. Large investments were made in breakwaters which were for the most part ineffective or incapable of surviving under tidal action. Present practice is to heavily brush the banks where undercutting takes place in an effort to guide the water rather than to re-direct it. Results look promising but there is still much to be done before this problem can be fully solved.

Aboiteaux are built with the aid of draglines and bulldozers. The stream bottom and banks are heavily brushed prior to beginning fill operations and the fill is built up in layers of mud with brush mats every 12 to 18 inches apart vertically. The brush mats are securely anchored by heavy cross staking, the stakes being from 6 to 8 feet long. The early builders usually placed the sluiceway in the middle of the channel. Now it is sometimes found desirable to place the sluiceway in an independent dry excavation in order to avoid intensive run-back flows when the tide recedes.

Aboiteau construction has to be observed first-hand before one can fully realize just how difficult and precarious the work is. Aboiteaux as now constructed are usually finished to 2:1 or 3:1 slopes. In the past they were built with almost vertical slopes. The old conventional facing was salt marsh sod but to-day the largest aboiteaux are faced with gravel and rock; this appears to be much more practical.

The aboiteau is considered analagous to a reinforced concrete structure, marsh mud being the aggregate, brush being the reinforcing steel and stakes the stirrups. Undoubtedly much of the load of the structure is carried into the banks by arching action rather than into the stream bed where foundation conditions are often extremely poor.

The major soil mechanics problems in aboiteaux construction are:

1. to determine the actual strength of the reinforced fills in order to design larger structures and eliminate great lengths of dike and expensive stream bank protection;

2. to investigate the feasibility of other types of structures.

In concluding, it may be mentioned that in some special locations the standard earth core dam may be used. One of these has been installed. No problems of special interest were encountered during the course of its design and construction.

Discussion

In reply to a question from Mr. Knight regarding the function of gates and sluices, Mr. McCarthy said that the sluices let the fresh water out and that the gates prevent the sea water from entering. This is accomplished by an automatic flap-type valve.

Mr. Peterson outlined present practices in the construction of aboiteaux, saying that they consisted alternatively of brush mats and 18 inches of mud. Now flatter slopes in the order of 3:1 are being built and it is felt that they should be analyzed mathematically and that factors such as "pull out" and strength of the brush should be determined. Mr. Peterson said that such studies would be extremely complicated and that the data presented would be of doubtful value. Nevertheless, he asked for the opinions of those present on such a study. It was the general opinion that such an investigation would not be of any great use and that present practices -- those used for many years -- should be continued.

Mr. Keiller suggested that concrete cribbing might be used to avoid undercutting in certain locations, but it was pointed out that the bearing capacity of the marshland mud was low; therefore this would not be suitable. The unit weight of aboiteaux is 80 to 90 lb. per cubic foot.

Mr. Hall said that brush for diking purposes was used extensively in Europe and it was found that when the brush was kept wet it was a more permanent construction than concrete. Mr. Hall asked Mr. McCarthy if he had obtained information from other countries on this use of brush. Mr. McCarthy replied that information had been procured but had been used without success.

Section 7

Soil Temperatures and Frost Penetration

by

C. B. Crawford

The problem of understanding soil temperatures, frost penetration and general frost action has existed for many centuries. One of the first records of soil temperature measurements was published by Professor Forbes of the University of Edinburgh in 1846. Later advancement in instrumentation resulted in more widespread measurements. In 1896, Professor Cal endar of McGill University published the first of his series of papers on soil temperatures and in 1910 Professor Bouyoucos of the University of Michigan began an extensive investigation of soil temperatures. In this work he posed many questions which are still unanswered.

Activity in the investigation of this problem has continued through the years and during the last decade has achieved considerable prominence. No doubt this is due in part to the requirements of modern design of highways and airports. Also, in recent years waterworks engineers have taken an active interest in the prediction of frost penetration. Besides these two main fields of engineering an understanding of the thermal regime in soils is required for the design of flat slab foundations, the installation of heat pumps, the placing of power cables, the construction of cold storage plants, and for founation work in permafrost areas. In each of these fields the effect of cold weather is serious in the Canadian climate.

The complexity of the problem can best be illustrated by considering the variables which affect soil temperatures. These factors may be divided into two types, external and internal. The external variables include all the meteorological factors such as air temperature, sunshine, precipitation, wind, humidity, and vapour pressure. The internal factors include nature and type of ground surface, specific heat and thermal conductivity of the soil, radiati n, soil moisture content, organic content, texture and structure, salt content, evaporation, and moisture movement. In our theoretical analysis all external factors except air temperature are usually neglected and it is often necessary to neglect many of the internal factors.

Investigations by the Division of Building Research, National Research Council

The soil temperature project of the Division has included only an investigation of soil temperature variation and frost penetration and not the whole field of frost action. In this study, temperatures have been measured during the last three years in test pits which were backfilled with sand and clay. The field project consisted of four clay pits and four sand pits. Of each group, two pits were densely backfilled, two were loosely backfilled, and one dense and one loose pit were cleared of snow during the winter. In this manner an attempt was made to study the effect of four variables: air temperature, soil type, density, and snow-cover.

In addition to the test pits, temperature measurements were recorded for a period of three years under streets in the City of Ottawa. Also, records were kept by the Water Works Department of frost depth and cover conditions at every location of frozen water mains in the City.

This project may be extended in the future to include the collection of frost penetration data throughout Canada, together with a study of meteorological records.

The investigation has not yielded all that was hoped for owing to recent mild winters. Some tentative conclusions have been made, however, regarding the variables which are under study. The results have shown that frost penetration is reduced by approximately 2 feet for each foot of loose disturbed snow. Frost penetration has averaged about $1\frac{1}{2}$ times as deep in sand as in disturbed clay. Disturbance of clay soils increased frost penetration by 50 per cent. The true effect of density has not been clearly determined as yet. Additional information has been gathered which relates frost penetration to freezing index.

<u>Investigations by the U.S. Corps of Engineers</u>

The U.S. Corps of Engineers have made studies of frost penetration at 15 airfields for the purpose of establishing design methods for airfield pavements on frost-susceptible soils. The field work has been accompanied by laboratory tests and theoretical and mathematical studies of heat flow. This work which is presently restricted has resulted in several equations for predicting frost penetration and a simple relation between frost penetration and freezing index.

The investigation has not been entirely successful. The Corps of Engineers still recommends a non frost-heaving base-course to the depth of the expected frost penetration. No improvement has been made on the grain size criteria for determining frost-susceptible soils. The investigations included an extensive study of the thermal properties of soil but in these studies the effect of thermal moisture migration was neglected and it is considered that the results are therefore open to question. Some study was made of the leaching of admixtures, the loss of bearing capacity during spring breakup and the effect of cover in reducing frost penetration. Work is continuing on the effect of particle size, compaction, void ratio, permeability, capillarity, and on the effect of the position of the groundwater table on ice segregation.

Other Work

At the present time, the Committee on Frost Heave and Frost Action in Soils of the Highway Research Board is attempting to promote work on frost problems at universities and state highway departments. In preparation for this, an extensive review of the literature on frost action was made for the Committee by Mr. A.W. Johnson of the Highway Research Board and by Mr. William Lovell of Purdue University. In addition to this work, studies of frost penetration and frost action are being made by some state highway departments and by the Ontario Department of Highways.

Problems

Frost action problems have not yet been completely solved although much progress has been made. It is still not possible to forecast frost penetration with certainty. The relation between frost penetration and freezing index has probably been over-simplified. It is thought by some that the rate of accumulation of degree-days of frost has considerable bearing on the frost penetration. The true effect of snow-cover has not yet been definitely determined. Probably the most important lack of information is in the understanding of the effect of moisture and moisture migration. Dr. N.B. Hutcheon of the Division is of the opinion that a complete understanding of the process of moisture migration would solve many problems concerning frost action. Possibly the continuing collection of field records together with a fundamental study of moisture will yield answers that have been sought for many years.

Discussion

Mr. Longley commented on the use of the term "freezing index" and asked if Mr. Crawford found it to be a workable term and also if only the temperature below freezing was measured. Mr. Crawford replied that the freezing index was cumulative, i.e. the temperatures above freezing were positive and those below freezing, negative, and the algebraic sum of the two was taken as the freezing index. Mr. Longley said that the negative values were not generally used and Mr. Crawford explained that the Division of Building Research investigation had followed the procedure used by the U.S. Corps of Engineers. Mr. Longley suggested that "degree days" should be the term used. Dean Hardy mentioned that in his soil temperature studies, Dr. A. Casagrande had used the term "degree hours".

Dean Hardy said that at the Highway Research Board symposium on frost action several representatives of state highway departments reported that in correcting a frost boil area, they did not remove the frost-heaving soil to the full depth of frost penetration. The Corps of Engineers follow the same practice. Dean Hardy mentioned that in the Proceedings of the Second International Soil Mechanics Conference a paper from Denmark stated that frost heave was not damaging if the material was not susceptible to ice segragation to a depth of 60 per cent of the frost penetration.

The Chairman said that he thought the term "freezing index" was an over-simplification. He mentioned the winter of 1947 in Ottawa where, for a time, there was very little snow and the weather was extremely cold. "Freezing index" would not be adequate to cover this type of situation. He mentioned that the Division of Building Research had a climatologist on its staff and that matters such as this would be studied.

Dean Hardy asked if anyone had information on the penetration of the frozen zone after the surface had thawed in spring; this had caused pipes to freeze at a depth of 12 feet in July. In the ensuing discussion, the Chairman said that it was generally known that the time lag of temperature at a depth of 15 feet was 6 months. Mr. Harwood said that in Russian literature on soil temperatures, this time lag was considered as an energy wave.

This ended the technical discussion of the first day; the remaining time was devoted to arranging the program for the second day.

SESSION OF JANUARY 11, 1952

Dean R. M. Hardy served as Chairman for the first session of this day's proceedings.

Section 8

Heaving of Curling Ice Sheets

by

B. B. Torchinsky

As is generally known, Mr. Torchinsky said, curling is fast becoming one of the most popular winter sports in Western Canada. It is played on an indoor sheet of ice, about 14 feet wide and 140 feet long. A prime requisite for the game is that the ice sheet be kept perfectly level. This is usually maintained by flooding the sheet as often as necessary, sometimes two and three times per week.

Most curling clubs have eight to ten ice sheets, and if these tend to heave often and badly, they become a constant source of trouble and inconvenience. Many inquiries have been received by the University of Saskatchewan regarding ice heaving and how to stop it. As a result of these inquiries it was felt that an attempt to analyse the difficulty was justified.

An examination of the conditions existing at those rinks in Saskatoon which were most troubled with heaving ice indicated that the major cause of the trouble was ice segregation and formation of ice lenses in the underlying soil, causing upward heave of the ice -- similar to frost heaving in highways.

In the winter of 1949 a series of thermocouples were installed in several sheets of a Saskatoon Curling Club at depths of from 1 to 5 feet below the surface. Subsequent readings of these thermocouples throughout the winter season revealed that the frost line penetrated to a maximum depth of from 4 to 5 feet. At the particular rink investigated, the water table was about 6 feet below ground surface. The underlying soil was a clayey silt, having about 27 per cent of its particles less than .02 millimeters in diameter. Thus, ideal frost heaving conditions were present and ice heaving in this rink was very bad. In the fall of the year 1951, a 2-foot layer of silt was removed from one of the worst sheets and replaced with clean, fine gravel. The surface voids were filled with a thin layer of fine sand. Precise level measurements have been taken throughout the season on this sheet and several others. These have shown that the treated sheet is now the best one in the rink, showing very little movement up to the present time. This then, does illustrate that frost heave is the major cause of ice sheets not remaining level, and that removal of one of the necessary conditions for frost heave to take place can be a satisfactory treatment. Further precise level readings will be taken until the end of the season to see whether or not the treated sheet will start to heave when the frost line penetrates below the 2-foot thickness of gravel into the frost-susceptible material.

Discussion

Mr. Fairbairn asked if lignosol had been used to treat such heaving and Dean Hardy replied that it had been tried in Edmonton with fair results. Investigations were carried on over a 3-year period and as soon as there was evidence of heaving, lignosol was injected and heaving was stopped at that location. In another case, Dean Hardy said that 2,500 square feet of an ice rink had been treated with lignosol in the fall and although there was more movement than had been expected, it was considered that the lignosol had been fairly effective. Dean Hardy said that injecting the lignosol was the main problem with this method of treatment.

Mr. Peterson asked what methods of injection had been used and Dean Hardy explained that the method was essentially one of grouting, the 40 per cent lignosol solution being pumped through a pipe, perforated at desired depths, at a pressure of 30 lb. per square inch. The path followed by the lignosol after injection seemed to be mainly a horizontal one and one of the main difficulties had been in the channelling action of the lignosol.

Dean Hardy had mentioned that the injection pipe was sealed at the top and Mr. Trow asked for more details on this. Dean Hardy replied that both augers and packing had been used in grouting attempts without success but a high-pressure grouting method employing burlap sacking on the pipe had been tried with considerable success. He mentioned that the U.S. Bureau of Reclamation had used up to 100 lb. per square inch with this method.

Replying to a question from Mr. Trow regarding the permanence of lignosol, Dean Hardy said that he would be able to answer this more fully in a few month's time when he would have data on three locations of the Canadian National Railways which had been treated. From preliminary results he could say that out of the three locations, one seemed to have stopped heaving except for the expansion of water to ice. In the second location, the amount of heaving had been reduced but shimming of rails was still necessary. In the third location lignosol had little or no effect; heaving in the order of 4 to 5 inches had taken place coinciding with cold spells. Dean Hardy again mentioned that injection was one of the main problems and said that they had never been able to inject the required amount of lignosol to achieve maximum effect according to laboratory tests. He said that Professor Hurtubise had treated portions of the Chicoutimi highway and that heaving had been reduced by 50 per cent. The amount of lignosol injected, however, was not enough to give the maximum results and after the treated areas thawed, the soil characteristics were altered.

Regarding injection, Mr. Torchinsky asked if the lignosol is pumped into the soil right underneath the frozen zone in the soil. Dean Hardy said that lignosol has to be injected into the area where the frost will be for the greatest length of time. He said that it must be injected well below the frost penetration line, as once the soil is frozen the effect of the lignosol is lost.

Concerning the cost of using lignosol, Dean Hardy said that on the Chicoutimi highway investigation, the cost of one treatment was about one-half the cost of removing the frost-heaving material and replacing it by granular fill. He said he did not know whether the lignosol method was cheaper than removing the soil as it was all related to the permanence factor. Lignosol is used in emergencies where cost is not the main factor but from the long-range point of view, Dean Hardy said he could not express an opinion.

Mr. Ripley asked through which company lignosol was available and Dean Hardy said that Lignosol Chemicals in Quebec City were the manufacturers. He said that the properties of lignosol which enable it to stop ice segregation are not peculiar to lignosol. Any wetting agent would produce the same results and some work is being done to see if other agents have more suitable characteristics. Since costs must be small, one advantage of lignosol over other such materials is that it is a waste product. Dean Hardy said that a lot of research on lignosol had been done, notably by the Pulp and Paper Research Institute of Canada and by the Eastern Paper Company, but surprisingly enough, there was still not much known about it.

As lignosol was mentioned as an emergency treatment the question arose if it would remove an existing heave. Dean Hardy said that it would not have this effect but the heaving would be stopped as of the day of the lignosol injection; thus it might stop heaving before it became too great a hazard. This situation was especially applicable to airfield runways.

Section 9

Soils and Foundations Work in Manitoba

by

A. E. Macdonald

In speaking of some soils and foundation work in Manitoba, Dean Macdonald reminded the meeting that Winnipeg is part of the site of old Lake Agassiz which once attained a depth of some 500 feet over the area. Under the business district are some 50 to 60 feet of clay usually with some silt at shallow depths; then about 5 to 10 feet of glacial drift, referred to locally as "hardpan" above limestone bedrock. The top clay strata are usually mixed and unstable and a yellow silt stratum has generally been a cause of differential settlement troubles for foundations which have been built above or on it. Heavy buildings have been founded on concrete piers in open wells to bedrock, or on concrete piers belled out on the hardpan. Lighter buildings have been satisfactorily supported on spread footings in the clay, provided the footings have been carried to the relatively stable brown, or preferably blue, clay below the upper silt and disturbed clay strata, and provided that a conservative allowable bearing value has been used along with a careful proportioning of footing sizes to minimize differential settlement which would otherwise occur in these plastic and "sticky" clays.

Some 20 years ago, equipment was developed locally which would bore a hole a foot in diameter or larger through these clays to the glacial drift formation, and power operation of these flat augers for doing so has been improved since that time. These bore holes are made without benefit of shoring. They may be made up to several feet in diameter without difficulty. As the plastic clays usually contain odd stones and boulders which would normally stop operations of a conventional post-hole auger, it is possible at times to bring these stones up on the flat type auger. Otherwise the bore hole may be enlarged if necessary with a larger diameter auger and a workman lowered down the hole to slip a noose around the boulder and so remove it. This equipment can be used for boring exploratory unshored holes for soil sampling and for taking undisturbed samples; for boring unshored holes to hardpan for cast-in-place piles or piers, the latter being uniform in bore or else manually belled out at their base up to twice the bore diameter; and for boring unshored holes to hardpan for underpinning foundations. In the latter case, universal joints are used to connect sections of the operating line in order to place the holes, if desired, staggered or along the centre line of the foundation wall. This equipment, therefore, is universal in performance and is both simple in operation and economical in cost.

Mention was made of a problem which sometimes develops when boring operations penetrate through a thin stratum of clayey-silty-sand overlying the hardpan where water under pressure below this stratum may rise quickly to a considerable height in the hole. A small brick apartment block near the Assiniboine River was underpinned in 1937 and these conditions were encountered. A short bullet-nosed wooden plug was driven through this stratum, and the remainder of the hole then filled with concrete. No visible signs of movement have since appeared.

Short cast-in-place piles, bored 12 or more feet from the ground surface into stable brown clay and then spread or reamed at the base by means of a special type of auger are used.

Caution is necessary in the indiscriminate use of portland cement for concrete for foundation work in the general Winnipeg area. Sometimes sulphate or "alkali" is found in sufficient concentrations to disintegrate ordinary concrete in a relatively short time. Kalicrete cement has been developed in Canada for such areas of high sulphate concentration and its use is recommended.

A number of pictures of foundation investigations and of soil sampling operations which have been carried out during the past year at various points in Manitoba were shown and described.

Discussion

Dean Hardy remarked that the matrix in boulder clay is a silt and is a distinct deposit which, when saturated, is unsuitable as a foundation material. Dean Macdonald said that in the Winnipeg area the till did not constitute a ridge but rather a definite till plain.

In discussing the use of alkali-resistant concrete, Dean Macdonald mentioned that the testing laboratories had taken soil samples to determine the extent of alkali. The location of alkali was found to be somewhat "spotty" but for safety's sake Kalicrete was specified as a general rule.

Section 10

Foundation Investigation in Winnipeg Following the 1950 Red River Flood

by

A. Baracos

During the Soil Mechanics Conference held by the Associate Committee on Soil and Snow Mechanics in 1950, the author reported on a preliminary investigation of the effects of the 1950 Red River Flood on foundations in Winnipeg (1). At that time it was apparent that a large part of the damage had occurred, during and immediately after the flood, to inadequately designed and poorly constructed shallow foundations. In certain areas soil swelling had occurred during the inundation and had caused additional damage due to differential movements. The occurrence of these movements, and the knowledge that soil moisture changes had previously caused damage to foundations in Winnipeg, prompted further study.

Soil Conditions, Flood Information

Briefly, the soils in the Red River Valley are silts and clays some 40 to 50 feet deep, deposited in the bed of the former glacial Lake Agassiz. The material is highly stratified and the clays have given considerable foundation trouble, shrinking excessively during dry years and swelling when the soil moisture has increased. A number of detailed reports have appeared on foundation difficulties in Winnipeg (2, 3, 4, 5).

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(1) Baracos, A. Soil Mechanics and the Winnipeg Flood, Proceedings of 1950 Soil Mechanics Conference, Associate Committee on Soil and Snow Mechanics, Technical Memorandum No. 19, National Research Council, Ottawa.

(2) Riddell, W.F. "Foundation Conditions in Winnipeg and Immediate Vicinity", Proceedings of 1949 Civilian Soil Mechanics Conference, Associate Committee on Soil and Snow Mechanics, Technical Memorandum No. 17, National Research Council, Ottawa.

(3) Macdonald, A.E. "Report of the Winnipeg Branch of the Engineering Institute of Canada Committee on Foundations", The Engineering Journal, November, 1937.

(4) Fosness, A.W. "Foundations in the Winnipeg Area", The Engineering Journal, December, 1926.

(5) Allaire, A. "The Failure and Righting of a Million-bushel Grain Elevator", Transactions of the American Society of Civil Engineers, Vol. 80, December, 1916.

Only a brief mention of some of the factors causing the flood are within the scope of this paper. These were heavy rainfall during the summer and fall of 1949, continued and severe frost prior to the snowfall, and the late and sudden spring thaw accompanied by heavy rains. These not only contributed to the flood but also caused a general increase of soil moisture in non-flooded areas.

Early in April of 1950, the level of the Red River in North Dakota and parts of Minnesota began to rise and by mid-April the river was flooding villages and cutting communications. Flood warnings were issued in Winnipeg on April 11 and by April 20 the flood stage had been reached. Records kept by the City of Winnipeg Engineering Department show that for 51 days the river remained above flood level, and at the peak of the flood was 12.9 feet above this level.

Approximately 8,200 houses were directly affected by the flood waters in Greater Winnipeg. Backing of sewers also brought water into many basements in non-flooded areas.

Investigation Procedure

Investigation procedure and results of the early post-flood period were reported at the 1950 Conference. Since moisture changes and accompanying soil volume changes were known often to be slow processes, it was decided to continue the investigation over a longer period of time. The objects of the prolonged study were:

(1) To determine if movements took place in 25 buildings selected in both flooded and non-flooded areas during a period of 15 months after the flood. Both damaged and undamaged buildings were selected;

(2) to determine, if possible, why soil swelling took place almost immediately during and following inundation;

(3) to determine why soil swelling had occurred more in some areas than in others. A program of test holes with visual soil classification was carried out;

(4) to study soil moisture variations in different areas of Greater Winnipeg. Test holes were augered periodically in 9 different locations during the 15-month post-flood period and soil moistures determined.

Results of the Investigation

Periodic visits to the 25 buildings selected for study showed that, in general, a slight subsidence occurred where swelling had previously taken place. A notable exception was a large gymnasium where additional swelling took place. An increase in the moisture content of the supporting clay was found and may have been caused by new construction in the area.

Examination of clays under foundations and basement floors that were removed following damage due to soil swelling in all cases revealed very high moisture contents. Soils near weeping-tile drains were especially wet and soft. These drains provided a path for moisture to the soil when water backed into them during the flood. Clays taken into the laboratory and dried were found to crack and laminate extensively. Rapid penetration of water into such clays may be explained by the presence of shrinkage cracks and laminations which were reported in earlier reports to be present as a result of dry years. Once swelling occurred and sealed the cracks, moisture removal would be a slow process, depending on the permeability of the material, which explains the slow settlement of the heaved areas.

Test holes showed that large portions of the flooded areas were fluvial sands and silts. This was contrary to the previous information that the entire area consisted of the highly plastic clay and silt deposits of Lake Agassiz. Changes in the course of the Red River had, over a great period of time, removed the clays and deposited coarser grained fluvial material in the river bed. Thus, low-lying areas were formed of better foundation material, but because of their lower elevation were more susceptible to flooding. The nature of the improved foundation conditions can be appreciated when the liquid limits and plasticity indices of typical Winnipeg clays and material from the low-lying areas are compared. Typical Winnipeg clays may have liquid limits as high as 120 per cent and plasticity indices as high as 80 per cent. In the low-lying areas liquid limits of 35 to 55 per cent and plasticity indices as low as 10 per cent were found.

The fortunate occurrence of the coarser fluvial deposits in the low-lying areas resulted in far less foundation damage than would otherwise have resulted. Swelling was limited to areas where the typical Winnipeg clays were inundated or their moisture content increased from the wet weather that preceded the flood.

Soil moisture contents decreased rapidly following the flood in the sand and silts in the low-lying areas. General but much slower decrease occurred in the clays to a depth of about 10 to 12 feet. Seasonal fluctuations caused some increases followed by decreases. The moisture content of the clay under the footings of the gymnasium mentioned previously were found to be exceptions. Moisture increases in this location were found to coincide with enlarged cracks in the walls of the structure.

Conclusion

Most flooded buildings were repaired during the year after the flood. In general, substandard foundations damaged by the flood were replaced using better construction techniques. Observation of flooded buildings and soil moisture tests, for a period of 15 months following the flood, indicate that no further damage from the 1950 flood is to be expected. What would have happened if more of the clay areas had been flooded is a matter of conjecture. From the examples of heave found in flooded clay areas, it is likely that foundation damage would have been far more extensive.

Section 11

Resistivity Methods of Soil Exploration

by

S. R. Sinclair

The term "resistivity" applies to one of the electrical methods of subsurface exploration which are included in the general category of "indirect". A method is termed indirect when soil samples are not obtained and positive identification of the strata requires correlation with the results of other methods of exploration. Thus resistivity is in the same class of exploration as "probing" or "sounding".

Field Methods

There are several resistivity field methods which may be used. One will be described. Four electrodes are driven into the ground in a straight line and at equal spacing. An electrical field is produced in the ground by means of the two "outer" or "current" electrodes. By measuring this current and the potential drop between the two "intermediate" or "potential" electrodes, the apparent resistivity of the soil to a depth approximately equal to the spacing of the electrodes can be computed.

In order to obtain a "depth profile", the centre point of the set-up is kept stationary as the spacing of the electrodes is varied. The usual procedure in Alberta has been to start at a spacing of 2 feet and expand the electrodes in 2- or 4-foot increments to a maximum spacing of about 80 feet.

Computations and Plotting

The theoretical analysis is based on the assumption that the current flow may be shown by the use of a flow net. The electrical resistance of the ground is computed in units of ohm centimetres or ohm feet.

Test results must be shown graphically. The most useful type of plot is one on arithmetic paper with cumulative resistivity as ordinate and spacing equal to depth as abscissa. The resultant curve is a series of straight lines with different slopes. A change in slope indicates a change in soil type or at least a change in electrical resistance at that depth.

Practical Application

The resistivity method has been used in Alberta with a considerable degree of success. The most reliable application is its use to determine the location of the boundary between two materials of vastly different resistance, e.g. clay and bedrock, or clay and gravel. The extent of gravel pits has been accurately determined by working radially outward from an existing pit of known quality.

Because it is indirect, the resistivity method of soil exploration is only reliable when used as a supplement to drilling and sampling. However, the practice in Alberta has shown it to be extremely helpful when used in this manner.

Discussion

Mr. Legget mentioned that work by resistivity methods had been done in Eastern Canada by various geophysicists. He said that a good source of information on this type of soil exploration could be found in the record of the recent American Society for Testing Materials symposium on soil reconnaissance.

Dean Hardy said that there was considerable literature available on resistivity methods, notably that issued by the U.S. Bureau of Public Roads and the U.S. Corps of Engineers. He said that resistivity methods for subsurface soil exploration have possibilities for foundation work in Canada which have largely been ignored. He considered that sometimes the results of resistivity testing were as accurate as the record obtained from bore holes and the cost was not exorbitant.

Section 12

Soil Description, Classification, and Symbols

Discussion led by R. Peterson

Mr. Peterson remarked that soil classification, soil description, and soil symbols had been discussed at previous Soil Mechanics Conferences. He suggested that Canadians as a body should endorse some established system in these three procedures. Accordingly he thought that the following should be standardized for Canadian work: a broad system of soil classification, a system of soil description, and graphical soil symbols.

As a start, he mentioned the memorandum by F.L. Peckover on the standardization of soil symbols issued in 1950 and recommended that the meeting should agree on the four basic soil symbols then proposed. Such an agreement would leave anyone free to make additions for his own special purposes if necessary, while retaining a standard framework.

Mr. Schriever thought that the Canadian group should certainly recommend a system of soil symbols. These could be used for all small jobs and enlarged as required for large undertakings.

Mr. Baracos said that he had tried using the proposed symbols and had found combinations of them cumbersome. He thought that the four basic symbols outlined in the report should be adopted as Mr. Peterson had suggested. Mr. Peckover agreed with this suggestion. He said that from the comments he had received on these proposed symbols, it was evident that the four basic patterns were in general satisfactory, but there was wide disagreement apart from this.

Regarding the adoption of the four basic soil symbols, Mr. Peterson said that if these were adopted in Canada, other countries might follow suit. Dean Hardy said that he did not object to their adoption but pointed out that their usefulness would vary depending on the person for whom the work was intended. If it was for a person with no experience in soils, symbols would be of no use in themselves and in a report the description of the soil would have to be written out in full. If, however, a soil mechanics organization was concerned then the symbols could be used to advantage. Mr. Lea was of the opinion that in such a case a poor standard was better than none. He said that his Company used a standard for the description of soils and found it extremely helpful.

In the course of the discussion, many alternatives were proposed. Among these were the use of a legend with a numbering system, and the use of colours with a legend. Finally, it was proposed by Mr. Peckover and seconded by Dean Macdonald that "the Canadian Soil Mechanics Group tentatively adopt the four basic soil symbols proposed in the 1950 report for use when desirable, with the proviso that they be used only as a supplement to a full written description of the soils; and, after a year's trial, that these symbols be considered again and comments received on their acceptability". After further discussion, the motion was carried 34 to 3.

The next subject considered was methods of soil classification.

Mr. Peterson suggested that the proposals put forward last year by Mr. Lea regarding soil descriptive terms should be accepted in principle.

Mr. Graves said that it had been suggested at last year's meeting that Mr. Peckover submit Mr. Lea's paper to American authorities and wished to know the results. Mr. Peckover reported that these suggestions had been submitted to the organizer of the recent symposium on soil description and classification of the American Society for Testing Materials but they had been too late for publication. He had been told that Mr. Lea's suggestions would be submitted to the proper American Society for Testing Materials Subcommittee at the earliest opportunity, but no further word had been received.

Mr. Peterson said that it had been hoped that the American Society for Testin, Materials would give guidance on these matters but this did not seem to be forthcoming.

Mr. Knight suggested that, as the matter was a controversial one, it should be considered by a Committee composed of members of interested organizations, who would present their findings at next year's Conference. A suggestion was made that this question might be discussed b the regional groups but it was the general view of those present that the setting up of a committee would be the wiser course.

Mr. Torchinsky was of the opinion that such a step would mean the needless loss of time. He thought it would be better to tentatively adopt a standard, try it for a year and then decide whether it was a desirable action. He put this in the form of a motion suggesting that "the Canadian Soil Mechanics Group should tentatively adopt the Casagrande system of soil identification as originally published in 1947 until next year's Conference and at that time decide on its general acceptability". Mr. Sinclair seconded this proposal.

Mr. Legget thought that serious consideration should be given to the uses of the words "classification" and "identification" when used concerning soil. He was strongly against the use of "classification" as, in the strict sense, soils were not being classified but rather identified. He suggested, therefore, that "identification" should be the term used in Canada. He also thought that Mr. Torchinsky's idea was a good one and that, if, after a year's trial, it was agreed that the Canadian group wished to adopt this as a standard, the Canadian Standards Association should be notified.

Those present voted on Mr. Torchinsky's motion and it was carried.

Section 13

The Neutron Moisture Meter

by

B. B. Torchinsky

During the summer of 1951 a con iderable amount of field work was done with the neutron moisture and density meter at the University of Saskatchewan.

The work on this meter was o iginally begun in the fall of 1950, based on suggestions by Dr. J. W. T. Spinks of the Chemistry Department, University of Saskatchewan. It was first reported to the National Research Council at their Annual Soil Mechanics Conference in Ottawa in December 1950.

The principle behind the neutron meter is that fast neutrons are rapidly slowed down and def ected in a medium containing hydrogen. A source of fast neutrons sur ounded by a detector foil ($indium^{115}$) is placed in the soil. The ast neutrons pass through the indium, affecting it very slightly, and into the soil mass. On striking the hydrogen nuclei contained in the soil moisture, the fast neutrons are deflected back as slow neutrons. These slow ne trons, n striking the indium foil cause it to become radio-active. For a fixed exposure time, and for a given strength of neutron source, the activity of the indium foil, as measured on a Geiger counter, can be calibrated against moisture conten .

After considerable experimentat on during last summer, the source chosen was a radium-beryllium source, having a strength of 50 milli uries (about 50 mg. of radium . The standard time of exposure was chosen to be 10 minutes, and the activity of the foil was ead at the 11th minute, with a portable count rate-meter (manufactured by Nuclear Instrument Co., odel 2610).

The same Ra-Be source is used f r density determinations, although for these, the gamma rays emitted by the source are made use of in place of the neutrons. The principle on which the density meter is based is that the number of gamma rays which are absorbed is dependent on the unit mass of the material into which they are directed. The Ra-Be source is lowered into the soil, with a thyrode (detector) tube above it. The thyrode tube is protected from receiving direct gamma radiations by a lead shield. Thus, the only rays which can strike it are those which have passed through the soil. The thyrode tube is connected to the por able count rate-meter, which indicates the rate at which gamma rays strike the tube. This value can be calibrated against soil density. All moisture content de erminations are accompanied by a densi y determination. This is required because the moisture calibrat on curve is most conveniently plotted as activity of indium foil vs. pounds of water per cubic foot of soil. Knowing the number of pounds of water per cubic foot of soil, and the wet density, the per cent moisture can easily be computed or picked off a simple chart.

During the summer, a considerable amount of field testing as done with the neutron meter. This work was done with the co-operation of the P.F.R.A., who provided laboratory facilities and personnel. A series of calibration tests were run at various locat'ons around the University campus. These were done in soils varying from sandy silts to alluvial highly plastic clays. These tests establishe definite calibration curves which were subsequently used in a series of irrigation tests. In these tests, several 2-inch di meter holes were drilled in a 12-foot square area to a depth of 10 feet and lined with aluminum casing. The area was dammed off so that it could be kept flooded with water. Moisture co tent determinations were made by oven-drying soil samples taken during the drilling of the test holes, and also with the neutron me r prior to flooding. These determinations compared within about 2 to 3 per cent moisture for values of water content varying from 15 to 40 per cent.

The neutron meter was used at various intervals during floo ing to detect moisture var ations. It indicated moisture content increases of from 15 to 32 per cent with the largest variations taking place near the surface, as would be expected.

At the conclusion of flooding, further soil samples were obtained by drilling holes close to the original cased holes. Water content determinations were run on these in the conventional manner, and once again, agreement with the neutron meter was found to exist to within about 2 or 3 per cent water content.

A second series of field tests were run on the neutron mete at the site of the proposed South Saskatchewan Damsite, in Central Saskatchewan. This site was chosen ecause the bedrock in this area is Bearpaw shale, a highly compressed, highl plastic clay which is very uniform and which has been tested carefully and in considerable detail by the P.F.R.A.

The results obtained in these tests were not as good as had been previously obtained. Out of 108 moisture determinations, 65 w re within 2 per cent, 26 were within 4 per cent and 17 varied by more than 4 per cent. All values refer to per cent water content. The average water contents were about 25 to 30 per cent. No definite reason has as yet been established for the lack of good correlation in the last series of tests. One possible explanation may be that some soil types have a hydroxyl ion in their lattice structure. The hydrogen nuclei due to this might affect fast neutrons in the same way as the hydrogen nuclei from water. This would mean that certain soil types would require individual calibrations to account for this.

Another possible use which has been suggested for the neutron meter is to determine the asphalt content of a bituminous mix. The hydrocarbons in the asphalt would provide hydrogen nuclei which could activate the instrument in much the same way as the water content of a soil.

The work on the neutron meter has been done by D. A. Lane for his thesis requirements for a Master's degree. Dr. Spinks has kindly provided assistance in the theories of nuclear physics. The Soil Mechanics Division of the P.F.R.A. has also assisted considerably in the field work. The work has been financed by grants from the Saskatchewan Research Council and the Prairie Roadbuilders' Section of the Canadian Construction Association. The above is all gratefully acknowledged.

Mr. Legget served as Chairman for the remainder of the meeting.

Section 14

General Remarks and Business

The Term "Soil"

The Chairman mentioned that agriculturalists and geologists took exception to the use of the word "soil" in soil mechanics work. He had investigated its use out of personal interest, and found a reference in the work of William Smith, dated 1835, where the term "soil" had been used in exactly the same sense as engineers use it to-day. In view of this precedent, the Chairman felt that engineers were quite correct in their use of the word. In view of the wide misunderstanding on the subject, he thought it might be worthwhile to bring this information to the attention of some publication such as "Nature". The question was discussed briefly and the suggestion was generally approved.

Mr. Baracos said that geologists used the term "unconsolidated material" as engineers use the word "soil", and thought that these two terms should be clarified.

Dr. Prest said that the word "soil", as used by geologists, includes all the material down to the place where it is still in the same form as that in which it was deposited. This, then, rules out such materials as silt and gravel. Dr. Prest suggested that the term "soil" was now so generally used in soil mechanics work that it would be useless to try to discard it. He suggested, however, that perhaps "soil profile" was a confusing term and that "soil contour" should be substituted in its place. However, Mr. Davis suggested that "soil profile" was already correctly used in highway work.

The Chairman thought that this sort of discussion was worth continuing and might be included in the agenda of next year's Conference.

Publication of Soil Mechanics Information

The Chairman thought that there was much valuable information on soil mechanics given at these meetings that should have a wider circulation than the proceedings of these meetings received. He suggested that all keep in mind the channels through which such information might be distributed. A note on the work on the neutron moisture meter had appeared in the September 1950 issue of the Canadian Journal of Technology and he recommended this publication as one means of publishing soil mechanics information. He also mentioned the British periodical Géotechnique which is solely devoted to papers on soil mechanics and geology. Its high standard of papers made it an excellent outlet for important contributions to soil mechanics literature.

The Chairman said that four papers on soil mechanics had been presented at the recent Congress on Building Research held in Britain and proposed that the A.C.S.S.M. issue these papers as a technical memorandum. All present thought this was a good suggestion and wished to receive a copy of this memorandum.

National Building Code

The Chairman said that the revision of this document was being handled by an Associate Committee of the National Research Council. This Committee guided the work of 10 technical committees, one for each section of the Code. The Chairman mentioned in particular the section on design of foundations and asked if everyone present would like to receive a copy of the draft of this section and send in their comments. All agreed with this suggestion.

"Building Research in Canada"

The Chairman mentioned this bulletin which is published by the Division of Building Research. It is designed to present an up-to-date picture of building research with particular emphasis on Canadian work. He said that a copy would be sent to anyone interested and he would be pleased to receive any comments upon it.

Local Soil Records

The Chairman mentioned that the Division of Building Research would be glad to assist in any way possible in the assembly of soil boring information in any area of Canada. The Ottawa Soil Mechanics Group had already initiated such a project in the Ottawa area. The Chairman hoped that such work would be resumed in Winnipeg and mentioned the work done by Dr. Prest in the Montreal district. He said that financial assistance would be available from the Council through the A.C.S.S.M. for the collection of such records as it was felt that this work would fill a definite need in many parts of Canada. These records were intended to include only boring and test pit logs and not physical properties of the soils.

Activities of Regional Soil Mechanics Groups

1. Toronto Group

Mr. D. R. Kempton, Chairman of the Toronto group for the current year, stated that three meetings had been held since the last Canadian Conference. In January, 1951, a talk was given by members of the Hydro-Electric Power Commission on field investigations, deep sampling, load testing and design. Mr. L. J. Chapman spoke at the February meeting on his development of a physiographical map of Ontario. In the fall, motion pictures were shown on the subjects of "Highway Soil Engineering", "How Tree Roots Can Damage Buildings", and "Building on Shrinkable Clays". These pictures were borrowed from the Division of Building Research.

Mr. Kemptón said that so far plans for 1952 included a meeting in January on grouting, one in February on frost heave of highways and small footings, and one in March on the data obtained from loading tests conducted by the Ontario Department of Highways.

2. Montreal Group

Mr. N. D. Lea, Chairman of this group, said that Professor Hurtubise, Chairman during the past year, was sorry that he was unable to attend this Conference.

Mr. Lea reported that two meetings had been held since the 4th Canadian Soil Mechanics Conference. In February, 1951, a joint meeting with the Civil Engineering Section of the Montreal Branch of the Engineering Institute of Canada was held on the techniques of field measurements. A fall meeting had discussed field permeability tests.

Two meetings were planned for the spring season. The first would be a discussion on the correlation of vane test results and laboratory tests. The other meeting would hear a talk on the geology of Pleistocene deposits on the Island of Montreal by Dr. V. K. Prest.

3. Ottawa Group

The Chairman of this group, Mr. Peckover, said that interest in group activities was increasing and average attendance at meetings had been about 25 persons.

During the past year, four meetings and two field trips were arranged. In January, Mr. Panter of the Ontario Department of Highways spoke on air photo interpretation as applied to the recognition of soil and rock types. In March, Mr. Knight had addressed the group on soil stabilization in highway practice. At the third meeting in April, a symposium on the geology and soils of the Ottawa area was presented with four speakers dealing separately with rock geology, pedology, physiography and engineering soil aspects of the area.

During the fall, the group visited Uplands Airport to see heavy compaction of granular materials and construction of concrete aprons and asphalt runways. Construction of a new collector sewer was also visited; rock and compressed air tunnelling methods were seen.

In December the most recent meeting was held to discuss the plans for the new season. In particular, a suggestion of Mr. McRostie's that the group undertake to collect local soil boring and excavation records, was discussed and a sub-group set up to work on this project. Mr. Peckover said that there was some enthusiasm on this subject and hoped that good progress could be reported at the next Canadian Conference.

Minute Books for Regional Groups

On behalf of the A.C.S.S.M., Mr. Legget presented bound minute books to the chairman of each of the active regional groups. He hoped that these would serve to keep a complete history of each group.

Funds for Regional Groups

Mr. Keiller asked about the possibili y of obtaining copies of papers presented at regional meetings. The Chairman replied that the question of reproducing such papers had been discussed during the past year, and the A.C.S.S.M. had suggested that limited funds be made available to each regional group to reproduce worth while papers or expend as they otherwise saw fit. If such papers were reproduced, it was hoped that they would be distributed to all Canadian members of the I.S.S.M.F.E. Mr. Keiller suggested that the Toronto paper on frost heaving be duplicated and Mr. Peterson suggested that the presentation to the Montreal group on penetration methods also be distributed.

Activities of International Society of Soil Mechanics and Foundation Engineering

At the Chairman's request, Mr. Peckover, secretary of the Canadian Section, reported on Society matters of general interest of the past year.

Response to the request to members to submit a report on their work in soil mechanics during the year was gratifying, and about 80 per ent of the members and their rganizations responded. These reports were compiled to make up the annual report from Canada to the Society (Technical Memorandum No. 22 of the Associate Committee on Soil an Snow Mechanics). Copies will be sent on request to any who have not yet received them.

Oth r member countries of the International Society also submit annual ports, in numbers sufficient to supply each member country with tw copies of each report. In Canada at the present time, one copy i ept on file and the other circulated to anyone interested. The names of those present who wished to review these reports were obt in d.

Mr. Pe ko er said that it was felt that by no means all of those actively interested in soil mechanics were enrolled as members of the Can dian Section of the Society. Accordingly, a canvass for more mem ers would be made in the near future, so that the Society mailing l st in Canada would e more complete and serve more efficiently t spread information of general interest. All those who were not members and wished to be informed of publications, and events such as the annual Soil Mechanics Conferences, were urged to join. No membership fees are involved at the present time.

Plans for the Third International Conference on Soil Mechanics and Foundation Engineering are now definitely going ahead for August 1953 in Switzerland. Excerpts from Bulletin No. 1, published by the Swiss committee in charge of arrangements, are included in this report as Appendix B. To these have been added information on the cost of travelling to and staying in Switzerland.

In closing, Mr. Peckover hoped that all who wished to attend, submit papers to, or order proceedings of the Conference would indicate their plans as soon as possible.

The Chairman asked that everyone consider seriously the possibility of submitting a paper for presentation at this important international gathering.

Next Canadian Soil Mechanics Conference

The Chairman thanked the Subcommittee on Soil Mechanics of the A.C.S.S.M. for arranging this Conference. He thought that perhaps every third year this meeting should be held elsewhere than in Ottawa and asked for opinions on this point. Mr. Knight thought that all Canadian conferences on soil mechanics should be held in Ottawa. Dean Macdonald said that he too thought that Ottawa was an excellent meeting place. However, if it was decided to hold the next meeting away from Ottawa, he suggested that it might be held in Winnipeg.

The Chairman thanked Dean Macdonald for this invitation. He said that at the present time all that could be said was that there would definitely be a Sixth Conference on Soil Mechanics in Canada, its place and time being left to the Subcommittee on Soil Mechanics.

Concluding Remarks

In conclusion, Mr. Legget said it had been a pleasure to meet with all those assembled. He thanked them for coming and for making the Fifth Annual Canadian Soil Mechanics Conference such a success.

APPENDIX A

List of Those Present at the Fifth Annual Canadian Soil Mechanics Conference

British Columbia

C.F. Ripley, Ripley and Associates, Vancouver.

Alberta

R.M. Hardy, Dean of Engineering, University of Alberta, Edmonton.

S.R. Sinclair, University of Albe ta, Edmonton.

Saskatchewan

B.B. Torchinsky, University of Saskatchewan, Saskatoon.

R Peterson, P.F.R.A., Saskatoon.

Manitoba

A.E. Macdonald, University of Manitoba, Winnipeg.

A. Baracos, University of Manitoba, Winnipeg.

Ontario

L.H. Bartlett, Department of Public Works, Ottawa.

G. Bird, Directorate of Works & Accommodation Department of National Defence, Ottawa.

W.R. Binks, Department of Resources & Development, Ottawa.

N. Burren, Department of Transport, Ottawa.

L.D. Boucher, Department of Public Works, Ottawa.

F.C. Brownridge, Ontario Department of Highwa s, Toronto.

J.W. Carmichael, Department of Public Works, Ottawa.

J.P Collins, Department of Public Works, Ottawa.

B.G. Craig, Geological Survey of Canada, Ottawa.

Ontario (Continued)

B.F. Cummings, Department of Public Works, Ottawa.

R.L. Egar, Department of Transport, Ottawa.

J.A. Elson, Geological Survey of Canada, Ottawa.

M.M. Davis, Ontario Department of Highways, Toronto.

E. Dumont, Department of Public Works, Ottawa.

E.L. Fowler, Department of Public Works, Ottawa.

H.M.G. Garden, Department of Transport, Ottawa.

E.V. Gilbert, Department of Public Works, Ottawa.

A.H. Graves, Department of Public Works, Ottawa.

T.A. Harwood, Defence Research Board, Dep rtment of National Defence, Ottawa.

I.W. Halpern, Directorate of Works & Accommodation Department of National Defence, Ottawa.

T. Hibling-Keiller, A.M.C./R.C.A.F., Beave Barracks, Ottawa.

O.L. Hughes, Geological Survey of Canada, Ottawa.

R.M. Johnson, Canada Cement Co., Ottawa.

W. Kalbfleisch, Department o Agriculture Ottawa.

D.R. Kempton, Hydro-Electric Power Commission of Ontario, Toronto.

J.A. Knight, Brunner Mònd Canada Ltd., Toronto.

J.R. Kohr, National Research Council, Ottawa.

A. Leahey, Department of Agriculture, Ottawa.

H.A. Lee, Geological Survey of Canada, Ott wa.

R.W. Longley, Meteorological Service, Toronto.

D. McIntyre, Department of Transport, Ottawa.

J.L. MacKean, Department of Public Works, Ottawa.

J.S. MacMillan, Department of Public Works, Ottawa.

G.C. McRostie, Consulting Engineer, 193 Sparks St., Ottawa.

Ontario (Continued)

H.C. Nixon, A.M.T.S. Branch (D.C.E.D.), R.C.A.F. Victoria Island, Ottawa.

E.B. Owen, Geological Survey of Canada, Ottawa.

T. Pascoe, Defence Research Board, Department of National Defence, Ottawa.

J.D. Paterson, Department of Public Works, Ottawa.

T.M. Podolsky, Geological Survey of Canada, Ottawa.

E.I.K. Pollitt, Geological Survey of Canada, Ottawa.

V.K. Prest, Geological Survey of Canada, Ottawa.

N.W. Radforth, McMaster University, Hamilton.

J.W. Reece, Department of Public Works, Ottawa.

R.P. Rowe, Department of Public Works, Ottawa.

R. Scott, Department of Transport, Ottawa.

D.W. Silliman, Engineering Department, City of Ottawa, Ottawa.

D.T. Smith, Department of Transport, Ottawa.

A.M. Stalker, Geological Survey of Canada, Ottawa.

J. Sutherland, Department of Transport, Ottawa.

B.J. Toms (Miss), Defence Research Board, Department of National Defence, Ottawa.

W.A. Trow, Hydro-Electric Power Commission of Ontario, Toronto.

E.B. Wilkins, Department of Transport, Ottawa.

Quebec

P.M. Bilodeau, Roads Department Province of Quebec, Quebec City.

D.F. Coates, McGill University, Montreal.

F.A. DeLory, Aluminum Co. of Canada, Arvida.

J.M. Fairbairn, Construction Borings Ltd., Montreal.

Quebec (Continued)

Per Hall, Foundation Co. of Canada Ltd., Montreal.

N.D. Lea, Foundation Co. of Canada Ltd., Montreal.

D. Maclaren, Maclaren-Quebec Power Co., Buckingham.

J.F. Mathys, Franki Compressed Pile Co., Montreal.

W.L. Pugh, Aluminum Co. of Canada Ltd., Montreal.

R. Quintal, Construction Borings Ltd., Montreal.

New Brunswick

H.W. McFarlane, University of New Brunswick, Fredericton.

Nova Scotia

L.W. McCarthy, M.M.R.A., Box 248, Amherst.

Division of Building Research, N.R.C. Ottawa.

R.F. Legget, Director

F.L. Peckover, Soil Mechanics Section.
W.R. Schriever, " " "
C.B. Crawford, " " "
W.J. Eden, " " "

APPENDIX B

Third International Conference on Soil Mechanics and Foundation Engineering

Switzerland 1953

In January, 1951, a poll of all National Committees conducted by the International Society of Soil Mechanics resulted in the selection of Switzerland as the meeting place for the Third International Conference. An organization committee for the Conference has been formed and is now preparing plans for the meetings in Zurich and Lausanne and excursions to be held August 16-26, 1953.

The first two international Conferences were held at Harvard University, Cambridge, Mass., in 1936 and at Rotterdam in 1948. The aims of such conferences are to encourage co-ordination of research programs and other engineering and scientific endeavours of the many countries represented, to advance international relations, and to establish personal contact between the members of the delegations. Members of the International Society will be the chief participants. In addition, other engineers active in soil mechanics or related fields will be welcomed and may register as guest participants.

The Conference is to open in Zurich August 16, 1953. During the first five days there will be eight technical sessions, each on a particular subject, lectures on Swiss problems and visits to the Soil Mechanics Laboratory of the Swiss Federal Institute of Technology. A special ladies' program will be arranged.

On August 21 a conducted excursion will start taking the participants through the Alps to Lausanne, visiting on the way places such as the earth dam of Marmorera, St. Moritz, St. Gotthard, the arched concrete dam at Grimsel, Interlaken, etc. The closing session will be in Lausanne on August 25 and will include the presentation of conclusions of the general reporters of the Conference and a visit to the Soil Mechanics Laboratory of the University of Lausanne.

For each of the technical sessions a general reporter will be appointed. His report will briefly review all important publications and recent developments in his field which have appeared since the last conference, including the papers submitted to the 1953 Conference. His report will be followed by discussion. The official languages of the Conference are English and French.

Approximate Cost of Travel and Stay in Switzerland

A wide range of travel and accommodation rates is available. Therefore the following figures, which are approximate but believed to be representative, are listed under minimum tourist and reasonable first-class rates:

Travel Montreal-Zurich return	Tourist	1st Class
by boat	$400	$700
by air	($600)*	$830

* Tourist transatlantic air accommodation planned by some American airlines with 30 per cent reduction on ordinary rates

	Tourist	1st Class
Registration at Conference	$12	$12
Hotel (Room, Breakfast, Service) 6 days	$26	$48
Meals - 6 days	$12	$30
Excursion (all included)	$50	$50
	$100	$140

1 dollar is approximately equal to 4.3 Swiss francs.

Proceedings

It has been agreed that the proceedings of the 1953 Conference will be limited to three volumes, of approximately the same size as the proceedings of the first and second Conferences. The first two volumes will contain the papers submitted to the Conference together with the eight general reports and will be mailed out prior to the Conference. The third volume will contain the records of the Conference, including the discussions and lectures.

Preliminary application for the proceedings, which will probably cost around 70 Swiss francs, or $17 Canada funds, may be sent to the Secretary of the National Committee.

Papers

The number of papers that can be accepted for publication in the proceedings is definitely limited, as can be seen from the reduced numbers of volumes to be published. At the same time an effort is being made to improve the quality of the papers and it is therefore proposed to print only papers which:

(a) are a real contribution to knowledge of the fundamental properties of soils,

(b) indicate new methods of application of soil mechanics which have proved satisfactory in practice,

(c) contain case records and measurements in connection with a detailed description of the soil and include comparisons between measurements and calculations.

All papers must first be submitted to the Secretary of the National Committee. The National Committee will then recommend the papers which in their opinion are most suitable for printing and forward copies of all papers to the Organization Committee and the General Reporters.

It is required that all who plan to submit papers should first submit a summary of not more than 300 words not later than 30th April, 1952. The actual papers will have to be submitted before 31st August. To all those who intend to submit papers, detailed suggestions as to length and manner of presentation will be sent.

In view of the active part which Canadians have taken in the first two international conferences, and the broad international interest in the forthcoming meeting, it is hoped that as many Canadians as possible will participate and that papers from Canada will form a valuable part of the Proceedings.

NATIONAL RESEARCH COUNCIL

ASSOCIATE COMMITTEE ON SOIL AND SNOW MECHANICS

LIST OF TECHNICAL MEMORANDA

No.	Date	Title
1	August, 1945	Proposed field soil testing device.*
2	September, 1945	Report classified "restricted"
3	November, 1945	Report classified "confidential"
4	October, 1945	Soil survey of the Vehicle Proving Establishment, Ottawa. *
5	November, 1946	Method of measuring the significant characteristics of a snow-cover. G. J. Klein. *
6	November, 1946	Report classified "confidential"
7	March, 1947	Report classified "restricted"
8	June, 1947	Report classified "confidential"
9	August, 1947	Proceedings of the 1947 Civilian Soil Mechanics Conference.
10	October, 1947	Proceedings of the Conference on Snow and Ice, 1947.
11	March, 1949	Proceedings of the 1948 Civilian Soil Mechanics Conference.
12	May, 1949	Index to Proceedings of Rotterdam Soil Mechanics Conference. (Soil Mechanics Bulletin No. 1).
13	June, 1949	Canadian papers: Rotterdam Soil Mechanics Conference.
14	December, 1949	Canadian papers presented at the Oslo meetings of the International Union of Geodesy and Geophysics.
15	April, 1950	Canadian survey of physical characteristics of snow-covers. G. J. Klein.
16	April, 1950	Progress report on organic terrain studies. N. W. Radforth.
17	August, 1950	Proceedings of the 1949 Civilian Soil Mechanics Conference.
18	November, 1950	Method of measuring the significant characteristics of a snow-cover. G. J. Klein, D. C. Pearce, L. W. Gold.
19	April, 1951	Proceedings of the 1950 Soil Mechanics Conference.
20	May, 1951	Snow studies in Germany. Major M. G. Bekker, Directorate of Vehicle Development, Department of National Defence.
21	August, 1951	The Canadian snow survey 1947-1950. D. C. Pearce, L. W. Gold.
22	October, 1951	Annual report of the Canadian Section of the International Society of Soil Mechanics and Foundation Engineering (June 1950 - June 1951).

* Out of print