Proceedings of the Eighth Canadian Soil Mechanics Conference

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PROCEEDINGS

OF THE

EIGHTH CANADIAN SOIL MECHANICS CONFERENCE

DECEMBER 16 AND 17, 1954

TECHNICAL MEMORANDUM NO. 36

Ottawa

April 1955

FOREWORD

This is a record of the Eighth Annual Conference of active Canadian workers in the field of soil mechanics which was held in Ottawa on December 16 and 17, 1954. A list of those in attendance is included as Appendix A. The conference was sponsored by the Associate Committee on Soil and Snow Mechanics of the National Research Council.

The meetings were held at the Building Research Centre of the Montreal Road Laboratories of the National Research Council. The morning of December 16 was devoted to two papers, the first presented by Mr. G.H. Johnston, and the second by Dr. G.G. Meyerhof. A short period of discussion followed the presentation of each paper. On the afternoon of December 16 a seminar on frost action was held, with contributions from Mr. J.A. Knight, Mr. E. Penner, and Mr. C.B. Crawford. A seminar on laboratory techniques was held on the morning of December 17, led by Mr. C.F. Ripley and Mr. P.J. Rivard. The afternoon was devoted to a general business discussion.

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Section I

Introductory Remarks

by R.F. Legget

Mr. Legget welcomed those in attendance on behalf of the Associate Committee on Soil and Snow Mechanics. A special welcome was extended to Mr. E.B. Eckel of the Geological Division, U.S. Department of Interior. Mr. Eckel was here in his capacity as Chairman of the Committee on Landslides of the Highway Research Board and attended the Second Meeting of the Canadian Landslides Subcommittee.

This meeting was sponsored by the Associate Committee on Soil and Snow Mechanics in order to promote the exchange of information among engineers engaged with soil mechanics in Canada. For this reason, two seminars had been arranged as part of the program, one on frost action and the second on soil mechanics laboratory techniques. It was hoped both would prove to be useful discussions.

Section 2

Site Exploration in Permafrost Areas

by

G.H. Johnston

The Mackenzie River system is one of the largest on the North American continent. The River has its source in Great Slave Lake and flows in a northwesterly direction for about 1,000 miles before emptying into the Arctic Ocean. More than 100 miles before discharging into the Ocean the Mackenzie River begins to divide into a series of channels that flow through its delta. The delta is approximately 50 miles wide by 100 miles long covering an area of more than 5,000 square miles. Unlike a typical delta, which fans out from its source, the Mackenzie delta is long and narrow. It is confined on the west side by the Richardson Mountains, a northerly extension of the Rocky Mountains, and on the east by the Caribou Hills and morainic hills, with extensive exposures of limestone, sandstone and shales.

Three main channels flow through the delta to the Ocean. On the most westerly of these, the Peel or West Channel, about one-half way down the delta, the present settlement of Aklavik is found. It is located on the inside or fill bank side of a sharp bend on this Channel. The buildings are strung out along the bank except for those built along the one main street extending back from the River.

Need for Relocation of Aklavik

About one year ago a decision was made to attempt to find a new location for the settlement. This decision was prompted by several important factors. A sewerage system and a water supply system are virtually non-existent and sanitary conditions as a whole are very poor. A proper installation on the present site would entail a very large expenditure of money. Contact with the "outside" is cut off for a period of 5 or 6 weeks during break-up and again in the fall during freeze-up. Thus, a year round airstrip for wheeled aircraft was considered essential but was not feasible at the present location because of soil and permafrost conditions. Anticipated expansion of the settlement would require far more area than is available now. The fringes of the town are on swampy, low ground which is not desirable. Further, the Permafrost Section of the Division of Building Research, during the summer of 1953 carried out a rather extensive soil exploration program in the town itself which showed that permanent construction would be very costly on the existing site.

Thus, by the first week in April of this year a site survey team was in the field in the Aklavik area. Prior to the field work, uncontrolled air photo mosaics were assembled in Ottawa by the Division and several tentative sites were selected from a study of these mosaics. These sites were then investigated by the survey team in the field. It was desired that these sites be seen under winter conditions and during the break-up period; this accounts for the early start of the survey. The sites selected for investigation were within a feasible moving distance of Aklavik and at approximately the same latitude.

Drilling and Sampling

Obtaining soil samples in frozen ground by conventional sampling methods is difficult and at times impossible. The principles of drilling and obtaining frozen soil cores are similar to those of diamond drilling. The Permafrost Section use a lightweight rig which is essentially a concrete core drill modified for permafrost work by the U.S. Corps of Engineers. A 5-foot NX size double-tube core barrel is used giving cores 2 inches in diameter. Drilling and sampling techniques are described in a recent publication "Permafrost Investigations at Aklavik, 1953", Technical Paper No. 16 of the Division of Building Research.

Though frozen ground has a resistance like that of rock, there are many differences in detail between drilling in frozen soil and in rock. Perhaps the greatest difficulty is that of trying to retain the core in its frozen state. In almost all cases the ice content of the soil is above the liquid limit of the soil so that when the core melts it turns into a soil slurry valueless for record purposes.

Speed is essential in sampling. When taken from the core barrel the core is cut into sections and representative portions are immediately photographed for a permanent record. The core is then described in the field notebook and samples taken for identification tests and moisture, or more correctly "ice content" tests. In sampling one has only 5 to 10 minutes before the core turns to a slurry.

Between 20 and 30 test pits were excavated this past summer in areas to which it was impossible to move the rig. At first they were allowed to thaw but because this was a slow process (a thaw of 3 or 4 inches every 2 or 3 days) and because of lack of time they were picked by hand to a depth of from 5 to 10 feet.

All soil samples were shipped by air to Norman Wells where they were tested in the Permafrost Research Station's small but adequately equipped laboratory.

Summary of Soil and Permafrost Conditions

During the 1953 exploration work at Aklavik 16 holes were drilled and sampled to depths up to 35 feet. Frozen ground was encountered from 8 inches to 24 inches below the surface depending on the thickness of the moss cover. The soil to the 35-foot depth is predominantly a series of stratified silts, fine sands and organic material. The complete absence of coarse-grained soils was striking for not even a pebble was encountered in any of the holes. The organic material ranged from black, hairline streaks to strata 2 feet thick. Plastic soils such as lean clays were scarce and when found were usually associated with organic deposits.

Ice segregation in the Aklavik fine-grained soils consisted predominantly of horizontal ice lenses up to 3/4-inch thick, although small and random vertical ice formations were observed. "Moisture", or more correctly "ice contents", in the first 6 feet ranged from 45 per cent to 340 per cent but averaged 140 per cent. Below 6 feet they averaged about 55 per cent. Thus the volume of ice, within the first 6 feet is 4 to 5 times the volume of soil. If the first 6 feet of soil were thawed and the water drained away then ground settlements up to 4 feet or 5 feet could be expected.

Results from these investigations may be considered as indicative of conditions existing in the delta as a whole. Therefore in the search for a new location, potential sites on the higher land bordering the delta to the east and west were investigated.

The Richardson Mountains to the west are comprised mainly of sandstones and friable shales of cretaceous age. A series of coalescing alluvial fans emanate from these mountains and link them with the delta. As would be expected the soils in the fans show the characteristics of the weathered portion of the mountains from which they are derived. Thus the predominant soil is silt sized with varying amounts of organic material and thin layers of "siltstones, or "mudstones", and some relatively more resistant sandstone pebbles. Horizontal ice lenses up to 3/4-inch thick formed the predominant ice segregation in the soil. Ice contents averaged about 85 per cent.

To the north of the alluvial fans, on the west side of the delta, low subdued rolling hills formed a distinctly different landform. Subsequent boreholes showed that predominantly silts and fine sands to a depth of from 12 feet to 17 feet overlay a compacted and rather dense glacial till. Between the fine-grained soil and the till a layer of 2 to 3 feet thick of a very stony sandy silt was encountered. Ice lenses up to 3/4-inch thick were evident in the silt but little or no ice was discernible in the till although it was well bonded.

Exploratory work on the east side of the delta showed the soils to be glacial in origin ranging from objectionable silt clays with much ice segregation to the more favourable gravels. Deposits of fine-grained soils were larger in extent in the areas investigated although extensive deposits of coarse-grained materials were also proved. The silts, fine sands, and clays contained stones up to 2 or 3 inches in size. The clay-sized soil was of low to medium plasticity and usually had high moisture contents, well above the liquid limits of the soil. Organic material was found in streaks and layers throughout. Ice segregation varied from closely spaced hairline lenses to deposits 15-inches thick. Gravels and sandy gravels were covered by organic material from several inches to several feet thick. Little ice was discernible in the gravels although individual stones and particles were coated, in some cases, with ice up to 1/32-inch thick and the gravel as a whole is well bonded.

Preliminary fieldwork during the summer of 1954 proved one site, on the east side of the Mackenzie River delta (known as East-Three), to be the most satisfactory. Further exploratory work was carried out in the area during the latter part of the season. It is on this site, approximately 40 miles due east of the present settlement, that the new Aklavik will be constructed. Without the aid of aerial photos and the knowledge of air photo interpretation and the use of a helicopter, the investigation of such a large area as the country adjacent to the Mackenzie delta for potential sites, would have been a long tedious job, almost impossible to complete in one season.

Discussion

Following Mr. Johnston's presentation, Mr. R.J.E. Brown with the aid of lantern slides illustrated details of the terrain of the Mackenzie River delta.

The Chairman Dean Hardy stated that the drilling work was part of the program for relocating Aklavik. He then introduced the team leader in the relocation work, Mr. C. Merrill of the Department of Northern Affairs and National Resources.

Mr. Johnston, in reply to a question from Mr. Graves stated that the frozen soil samples were obtained through use of a double-tube core barrel, the same type as used in hard rock drilling.

Mr. Baracos asked if Mr. Merrill would outline the organization of the relocation team.

Mr. Merrill replied that early in February the planning for field work was stated. A field team of eight were on the site by March 15th. The field team consisted of personnel from the Department of Northern Affairs and National Resources, Department of National Health and Welfare, Department of Public Works, Department of Transport and the National Research Council.

In the winter, helicopter was the main means of travel and proved to be almost essential. Dog teams were used to supplement the helicopter. In summer, water transportation was used.

Mr. Merrill stated that the use of air photos was essential to the job. The new site for Aklavik had to be accessible to water, have an all-weather airstrip and provide reasonable foundation conditions. The use of air photos had eliminated all but 2 square miles out of 5,000 square miles as possible sites. From the 2 square miles, eight sites were chosen for further investigation. It was interesting to note that the local residents did not suggest any alternate sites to the eight chosen by aerial photographs.

Mr. Merrill then asked Mr. Johnston if the ice segregation noted in the gravel might affect the bearing capacity upon melting.

Mr. Johnston replied that he did not think there would be settlement since only the odd particle had an ice coating. Because no undisturbed cores were obtained of gravel, this was based on observations of disturbed material from test pits.

In reply to Dean Hardy, Mr. Johnston could give him no information on the relative density of the gravel.

Mr. Peckover asked when and how the move of Aklavik will be made.

Mr. Merrill replied that the responsibility of the move rested with the Department of Northern Affairs and the Advisory Committee on Northern Development. It is now proposed that next summer access roads, a wharf, and temporary camp facilities, will be constructed on the new site. The first construction of new permanent buildings will begin in the summer of 1956. In the winter of 1956-57 buildings will be moved over the ice from the present site. By 1957 or 1958 the work of relocation should be completed.

In reply to Mr. Torchinsky, Mr. Johnston stated that all soil tests were conducted in a laboratory at Norman Wells. The testing was confined to identification tests. Frozen cores were photographed to have some record of their natural condition.

Mr. Johnston, in answer to Mr. Lea, stated that a piston-type pump was required because in case of seizure, a high pressure was necessary to free the bit. In drilling frozen ground a small flow of water is necessary for core recovery.

Dr. Radforth asked if the organic matter that was referred to as organic streaks, was residual matter that had been buried, or was it formed in place. Mr. Johnston thought it was organic matter which had been covered by subsequent deposition.

In reply to Mr. Robinson, Mr. Johnston stated that the thickness of the active layer varied from a few inches to 4 feet, depending upon location. The average thickness was about 18 inches.

Section 3

Correlation Between Penetration Tests and the Bearing Capacity of Cohesionless Soils

by

Dr. G. G. Meyerhof

Introduction

Because of the difficulty and expense of obtaining undisturbed samples of cohesionless soils, the bearing capacity of foundations in such materials is frequently determined from the results of penetration tests made on the site. The most important types of tests are standard (dynamic) and static penetration tests, both of which have been described in detail before (1).

By comparing the results of such tests on a number of sites in Canada and elsewhere, the Author has obtained an approximate correlation between them and a method of applying the results to an estimate of the bearing capacity of spread and piled foundations in cohesionless soils. A detailed account of this investigation will be published elsewhere (2), and the present paper merely gives a brief summary of this work.

Correlation Between Penetration Tests

Analysis of published test results, together with further evidence obtained at several sites in Eastern Canada, indicates that the standard and static penetration resistances are proportionate, and that the relationship between them can approximately be expressed by

$$q_c = 4N \qquad \ldots\ldots.1$$

where q_c = static cone resistance (tons per sq.ft.),
and N = standard penetration resistance (blows per ft.).

From this relationship the relative density of cohesionless soils, in terms of the cone resistance, can readily be obtained using the relationship published (1) between relative density and standard penetration resistance. Such an approximate correlation, including also the angle of internal friction of cohesionless soils, had previously been suggested by the Author (3) on the basis of a preliminary analysis of published data.

Bearing Capacity of Spread Foundations

It has previously (1) been suggested that the allowable bearing pressure (tons per sq.ft.) for a footing at some distance above the water-table is approximately given by

$$q_a = N/10 \qquad \ldots\ldots.2$$

and one-half the above value for foundations at or below the water-table. Similarly, for raft and pier foundations at, or below the water-table, it has been suggested (1) that the allowable bearing pressure is given by equation 2.

Hence, substituting equation 1 into equation 2, the allowable bearing pressure of spread foundation is approximately

$$q_a = q_c/40 \qquad \ldots\ldots.3$$

on the basis of the static cone resistance.

Comparison of equations 2 and 3 with the results of plate loading tests has shown that the estimates are frequently somewhat conservative and are sufficiently safe for the worst conditions encountered so far.

Bearing Capacity of Piled Foundations

Analysis of the results of penetration tests has indicated that the ultimate unit point resistance of a pile, q_c, may be expressed by equation 1, and that the ultimate unit skin friction (tons per sq.ft.) of displacement piles is approximately given by

$$f_s = \bar{N}/50 \qquad \ldots\ldots.4$$

or, for piles with small soil displacement (for instance, steel H piles),

$$f_s = \bar{N}/100 \qquad \ldots\ldots.5$$

where $\bar{N}$ = average standard penetration resistance (blows per ft.) along pile shaft.

Hence, using a factor of safety of 3, the safe load (tons) of displacement piles can be expressed by

$$Q_s = q_c A_p/3 + f_s A_s/3$$
$$= 4NA_p/3 + \bar{N}A_s/150 \quad \ldots\ldots\ldots 6$$

or, for steel H piles,

$$Q = 4NA_p/3 + \bar{N}A_s/300 \quad \ldots\ldots\ldots 7$$

after substituting equations 1, 4 and 5,

where A_p = sectional area (sq.ft.) of pile toe (point),

A_s = surface area (sq.ft.) of pile shaft,

and N = average standard penetration resistance (blows per ft.) near pile point.

Application of the above relationships (equations 6 and 7) to the results of pile loading tests has shown reasonable agreement.

The allowable load on a pile group is approximately given by the sum of the safe load on the individual piles (using a factor of safety of 3), provided the piles are not driven through nor underlain by compressible soils (1).

Conclusion

Comparison of the results of standard (dynamic) and static penetration tests has shown that a simple approximate relationship exists between them. This relationship enables either of the two types of tests to be used for determining the relative density of cohesionless soils and for estimating the bearing capacity of spread and piled foundations in such soils.

References:

(1) K. Terzaghi and R. B. Peck, "Soil Mechanics in Engineering Practice". J. Wiley, New York, 1948.

(2) G. G. Meyerhof, "Comparison between Standard and Static Penetration Tests and the Bearing Capacity of Foundations in Cohesionless Soils". Proceedings, American Society of Civil Engineers, 1955, Vol. 81 (in the press).

(3) G. G. Meyerhof, "Recent Studies of Foundation Behaviour". Journal, Engineering Institute of Canada, 1954. Vol. 37, p.123.

Discussion

Dean Hardy raised three questions regarding Dr. Meyerhof's proposed method of design:

1. Was there any maximum size limitation as to what might be considered sands and gravels?
2. Can the method be applied to saturated and unsaturated sands with equal validity?
3. Is the ultimate bearing capacity expression based on a shear failure or is it based on settlement?

Dr. Meyerhof replied that this method obviously could not be applied when the soil contained boulders. Since the penetrometer was 1 3/8 inches in diameter, the maximum particle size in which it could be used was 1/2 to 1 inch. Regarding the point on saturation, if a spread footing is located below the groundwater table, the bearing capacity will be one-half of that of the same footing located at some distance above the groundwater table. Since it is usual for piles to go below the groundwater table, this effect is taken care of by the measured penetration resistance.

As to the question of the bearing capacity obtained, the expression based on the penetration test is the ultimate bearing capacity at shear failure of the soil and has to be modified by a safety factor. With this safety factor, a safe bearing value is obtained. The minimum safety factor recommended is three, which is adequate for a small foundation. For large foundations, a safety factor of four or five is advisable at present in order to restrict the settlement. When more results have been assembled, it may be possible to reduce somewhat the factors of safety proposed.

Mr. Piette stated that the value of 1/30 of the cone resistance as design load seemed very conservative. He pointed out a case where an addition to an existing building was designed with a bearing pressure equal to 1/8 of the cone resistance. Since it did not seem to be a safe design, it was verified by a loading test.

Mr. Coates asked if the design method could be safely applied to glacial tills. He pointed out one example where penetration resistance indicated a good bearing capacity per pile. A subsequent loading test showed the bearing capacity of the pile as only 1/3 of that indicated by the penetrometer.

Mr. Legget asked if Dr. Meyerhof knew of any experimental evidence to validate the assumption that bearing capacity must be reduced by one-half below the groundwater table in cohesionless soils.

Mr. Millette considered particle shape to be important in the strength of cohesionless materials and asked if this was considered in the design method.

Dr. Meyerhof in answer to Mr. Piette stated that the design equations were admittedly conservative. He was hesitant to alter them at present because the method was based on only a few tests and limited experience.

In reply to Mr. Coates, Dr. Meyerhof considered that the formula could be applied to cohesionless tills. Referring to Mr. Legget's question, recent tests conducted in Great Britain had shown the assumption that bearing capacity must be reduced by one-half was correct. The results of these investigations would be published in Géotechnique in the near future*. Replying to Mr. Millette,

* G. G. Meyerhof. "Influence of Roughness of Base and Groundwater Conditions on the Ultimate Bearing Capacity of Foundations". Géotechnique, 1955, vol.5 (in press). See also G.G. Meyerhof. "An Investigation for the Foundations of a Bridge on Dense Sand". Proc. Third Int. Conf. Soil Mech. & Found. Engg., 1953, vol.2, p.66.

no work had been conducted to ascertain the effect of particle shape on penetration resistance. The relative density appeared to be the most important factor.

Mr. Coates asked if the standard penetration test could be related to penetration tests conducted with different sized hammers and different heights of fall. Dr. Meyerhof answered that theoretically they could be compared by pile driving formula to estimate the resistance equivalent to that of the standard penetration test.

Section 4

Seminar on Frost Action

Introduction - J.A. Knight

Mr. Knight pointed out that his talk would be an introduction to the problem of frost action, not necessarily including explanations and conclusions. Since frost action damage in this country is most evident on roads, he confined his remarks entirely to this topic. He pointed out that frost damage to roads is apparent in three phases; frost heave, frost boil, and frost break.

In beginning his talk Mr. Knight suggested that "cold", although a negative quantity of heat, be regarded as a positive condition for this discussion permitting the conception of cold penetrating the ground. He then pointed out that when the air temperature drops in the fall, and cold enters the ground, the moisture content usually is not large, perhaps 15 per cent. If this moisture expanded 10 per cent on freezing, it would only represent 10 per cent of 15 per cent or 1.5 per cent volumetric change. The lineal change would be the cube root of 1.5 or approximately 1 per cent change, which would produce a negligible vertical displacement. As the cold descends, however, water droplets form crystals and the growth of the crystals starts an entirely new series of forces permitting a continual growth as long as water is available and temperature conditions are suitable. The rate of penetration of the cold is reduced by the insulating effect of the ground and the release of latent heat. If the rate of penetration is fast, each crystal will be isolated by the underlying layers of soil being frozen and cutting off the water supply, but when the amount of cold supplied just balances the release of latent heat, the capillary water drawn to the crystal propagates the growth of ice masses. During the spring season, thawing commences from above but the water cannot escape due to frozen ground below and the upper part of the soil becomes thoroughly saturated.

Mr. Knight compared the condition of the saturated soil mass with quicksand. He suggested that the weight of traffic passing over a saturated mixture of soil and water causes pore water pressure which results in a quick condition that can be relieved only by a break in the surface forcing out a mixture of water and fines. This is the well-known frost boil. After further traffic,a hole beneath the pavement develops and the pavement collapses causing the fully developed frost break. To correct this situation, fine sand with a high surface area may be added to blot up the excess of water but this is not entirely satisfactory. As the season progresses the underlying layers of ice thaw allowing drainage of the excess of water the material becomes firm and the surface can be repaired.

Mr. Knight then noted that any frost condition can be cured if enough money is spent on it but that every case is an economic problem. If the same spot breaks every year, it is probably caused by trapped water in a rock pocket, or depression, or by a specific source of water such as a spring or porous stratum. To correct this situation excavation, drainage, and backfill, are required with proper regard to good material and good compaction. If the trouble is experienced in the same area but at different spots each year, bad soil condition over a considerable area is indicated and a general strengthening by additions of granular fill will be required.

Mr. Knight suggested that if crystal growth can be inhibited, the great increase of water in the soil and the build-up of frost lenses can be stopped. He believed that the addition of calcium chloride, in addition to its use in compaction and dust laying, would act as an anti-freeze inhibiting this growth. Although much qualitative information has been gained on the use of calcium chloride he said that his Company (Brunner Mond of Toronto) was interested in quantitative evaluation of the treatment and they are at present operating a fellowship at Laval University to achieve that end.

In conclusion, Mr. Knight emphasized two features related to frost damage. The first is that the amount of water in the soil in the fall has little bearing on the severity of the frost action and that the increase in water content does not come from the surface because the frozen soil is impervious, but comes from below and the amount imported depends on the temperature variations. The second feature is that all conditions can be corrected provided funds are available; the choice of method is dictated by economy.

A Program for Frost Action Studies - E. Penner

The rate of ice lens formation in soils is believed to depend on the ability of the soil mass to transmit moisture. This in turn is a function of its physiochemical properties. Whether an ice lens can be initiated depends on the absorptive force of water in the vicinity of the frost line. The total driving force is believed to be a combination of (a) the total temperature gradient and (b) the soil moisture suction gradient.

The first phase of the studies proposed by the Division of Building Research will be concerned with an investigation of the limiting pF at which ice lensing can be induced and the determination of soil moisture potential gradients under steady-state flow conditions in the absence of temperature gradients, except in the immediate vicinity of the ice lens. The flow rates induced by ice lensing will be compared with flow rates in an unsaturated permeability apparatus. The physiochemical properties are largely determined by the clay mineral fraction and will be described in terms of its type, specific surface area index, interlayer swelling, base exchange capacity and base exchange complex. Some identification tests will also be done as a means of describing the soil for classification purposes. The test specimens will consist, initially of artificially blended sand, silt, and clay fractions. Further studies will include the effect of normal temperature gradients. It is believed that such an approach will assist in the formulation of more adequate frost action criteria.

Climate in Relation to Frost Action - C.B. Crawford

Mr. C.B. Crawford, of the Division of Building Research, gave a brief account of a paper prepared for presentation to the 34th Annual Meeting of the Highway Research Board in January, 1955.

He pointed out the necessity of studying climatic factors together with fundamental and field studies of the frost action phenomenon in order to arrive at a satisfactory understanding of the problem. Previous climate studies were reviewed, particularly the attempt to combine air temperature and precipitation data as an index of the severity of spring break-up of the roads. In the present work an attempt is made to improve the "break-up index" by using not merely the value for precipitation but a value for soil moisture storage which is obtained by subtracting from the precipitation the amount of water lost in evaporation and transpiration.

Two examples of correlation of weather data to spring break-up were given at each of Calgary, Alberta and Ottawa, Ontario. Attention was drawn to climatic features which influence the degree of frost action such as the rate of accumulation and decrease of degree-days of freezing air temperature.

It was concluded that:

(1) the degree of spring break-up of roads is related to soil moisture conditions at the time of freeze-up and can be predicted reasonably well by an analysis of weather records;

(2) the use of a soil moisture storage factor is better than the use of precipitation data in establishing a break-up index;

(3) cold winters do not necessarily result in severe break-up; rather it is suggested that the rate of freezing and thawing is more important;

(4) in the examples cited there was no evidence that freeze-thaw cycling of air temperature affected the degree of frost damage to roads.

The importance of widespread observation and assessing of degree of break-up was emphasized in order to understand the effect of weather on break-up and to improve design.

Discussion

Mr. Knight commented that it was his experience that frost action or frost problems with roads depend more on the initial rate of penetration and not on the degree-days.

In reply to Dr. McLeod, Mr. Penner stated that the literature indicates that water could travel to the ice lens by (a) thermal gradient, and (b) in a form of liquid by suction gradients. It was believed that the latter is the most important mechanism.

Dr. McLeod considered that in view of the two possible means of moisture transfer, the use of an anti-freeze must not only depress the freezing point but it must also depress the vapour pressure.

Dean Hardy reported that several hundred freezing tests had been conducted at the University of Alberta. All the samples were saturated and they indicated that the water moved in the liquid phase.

Regarding Mr. Crawford's comments, it seemed to Dean Hardy that the crucial factor in the bad Calgary break-up was a gradual rise in water table over a number of years. Since the ice crystal must have water available, any index should incorporate these longer climatic cycles. Also if there is a severe break-up at one location, and if subsequent repairs are properly executed, this location will not break up in spite of a climatic index.

Mr. Baracos made some observations of ground movements at various levels. He had found that the surface layer actually shrinks in the Winnipeg area from December on, in spite of ice lenses growing in the surface layer.

Mr. Millette asked what effect such work as weed spraying would have on the accuracy of a break-up index, which was based on evapotranspiration.

Mr. Crawford replied that the Thornthwaite system of dealing with precipitation and evapotranspiration was developed for the classification of climate and would not take into effect such local factors as weed control. If more accuracy was required, then the Thornthwaite system would have to be modified.

Mr. Davis mentioned an experiment to study water phase movements in the soil. A soil sample saturated with salt water was subjected to a temperature gradient. After a time the salt was concentrated at one end suggesting that the water moved as vapour in one direction and returned as a liquid state in the other direction. In the field, he thought the method of moisture transfer would depend somewhat on the soil.

Mr. Davis concurred with Mr. Knight's observation that a rapid penetration of the frost line meant that break-up conditions would not be severe. Regarding the correlation of weather data, experience in Ontario had shown that a cold period followed by a thaw then a further cold period will cause large ice lenses near the surface. If there were more than one such cycle in the winter, a bad break-up could be expected.

Dean Hardy had observed a reduction in strength up to 50 per cent in soil which had been subjected to freezing for the first time in its history. Such a strength reduction was becoming more and more important with the increase in winter construction.

Mr. Knight asked if anyone knew the answer to the perennial question of whether or not a swimming pool should be drained.

Mr. Baracos reported that one swimming pool which had been previously drained and insulated with straw was being left full this winter and kept under observation. However, provision was made for emergency heating against the possibility of the water freezing to the bottom of the pool.

Mr. Beaudry stated that in Montreal it was usual practice to leave the pools full of water. Logs were put in the water, and there did not seem to be any danger caused by ice action.

Dr. Meyerhof referred to the problem of frost action under cold storage plants. In England, the detrimental effects had been controlled quite successfully by installing electric strip heaters under the cork insulation of the floor.

Mr. Crawford, referring to Messrs. Davis' and Knight's comments regarding the weather, stated that published records of such field observations were extremely scarce. He would like to see more field data published in spite of the apparently inadequate means of evaluating the severity of the break-up.

He agreed with Dean Hardy's comment that a change in the construction of the road might eliminate future break-up. But there were many secondary roads which broke up quite frequently and which could not be repaired properly because of lack of funds. Observations on such roads should be compared with a climatic index.

Mr. Penner asked Dean Hardy if lignosol had an anti-freeze effect much the same as calcium chloride.

Dean Hardy replied that lignosol acted as a dispersant and seemed to prevent the migration of moisture to the frost line. Any dispersant would accomplish this. Lignosol appeared to have a twofold action, (a) it reduced the permeability of the soil, and (b) it changed the surface tension characteristics of the water. Laboratory experiments had proven the effectiveness of lignosol, and the current problem is to develop an adequate method of introducing the chemicals into the soil.

In answer to Mr. Lane, Dean Hardy stated that because lignosol was water soluble, it was not a permanent remedy. In one field trial, there was some evidence of leaching away after four years. This problem could be solved by making lignosol insoluble by the addition of chromate salts. This formed a gel which in laboratory experiments had proven even more effective than the lignosol.

In reply to Mr. Robinson, Mr. Crawford stated that in the Ottawa area, it had been found that each foot of undisturbed snow would reduce the frost penetration by 2 feet on the average. Observations had shown a spread between 1.7 feet and 2.7 feet, presumably due to the variations in density of the snow. Indications were that pure ice reduced the frost penetration by an amount about equal to its own thickness.

Dr. McLeod suggested that the difficulty in evaluating the severity of the break-up could be overcome by use of a deflection test, such as the "Benkleman Beam Pavement Deflection Indicator" on the W.A.S.H.O. test road. This is a device which measures the surface deflection under the wheels of a loaded truck. The apparatus is simple and a test only requires a minute. Such a test would have to be used with caution in view of the variation in bearing capacity throughout the year. Such tests would have to be conducted at the minimum point of the seasonal bearing capacity curve.

Mr. Legget commented that the seasonal bearing capacity curve was the reverse of the seasonal groundwater level curve and suggested there may be some relation between the two.

Dr. McLeod stated that tests did not indicate any noticeable change in either moisture content or density in subgrades corresponding to the minimum point of bearing capacity.

Dr. McLeod thought that temperature measurements conducted on the W.A.S.H.O. test road indicated that the thickness of base did not change the time that the frost entered the subgrade. This applied to bases varying from 6 to 22 inches.

Dean Hardy stated that frost penetration is different in gravel than in clay. There are a number of factors which influence frost penetration, and it would be a mistake to neglect the effect of a thickness of granular bases as suggested by the W.A.S.H.O. test results.

Mr. Legget then reported on an interesting example of the control of frost action he had seen in New Hampshire. A skating rink had been built on the site of a former outdoor rink with natural ice. Later it was decided to install an ice-making plant. Investigation of the site showed the ice sheet to be founded on extremely frost-susceptible soil. This frost-susceptible soil was excavated and replaced with sand which was maintained in a saturated state. The depth of the excavation was determined by calculation, relating the latent heat of the saturated sand to the capacity of the refrigeration plant. The method had proven successful to date in preventing frost heave. This example suggests that it is the water, not the soil particles, which is important in the consideration of frost action.

In a discussion on frost heaving criteria, Mr. Millette suggested that possibly frost heaving criteria should not depend on particle-size analyses based on weight. For example, a gravel which was 75 per cent of the weight of a sample could represent a much smaller percentage of the volume and this could be the cause of an error in any criterion based on weight-size distributions.

Dean Hardy stated that in his experience he had found the Casagrande criterion quite reliable. Commenting on the theories of frost heaving, he thought that Taber's theory adequately explained most of the frost action that occurred in nature. Benkleman and Olmstead's tests could not account for large frost boils.

Mr. Penner did not consider the Casagrande criterion to be the complete answer. Soils which this criterion could classify as safe, according to recent literature, could support the growth of ice lenses. He agreed with Dean Hardy that Benkleman and Olmstead's theory of frost action was not accepted widely.

Dean Hardy stated that if the Casagrande criterion errs, it is because it is too conservative. Dirty gravel had been used quite successfully in the treatment of frost boils. There must be a certain amount of fine material in order to get ice segregation.

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Section 5

Seminar on Laboratory Techniques

Identification Tests, Compaction and Permeability - led by C.F. Ripley

In introducing the subject, Mr. C.F. Ripley stated that there were more or less standard procedures laid down for each test by the A.S.T.M. and other organizations. Since soil is a three-phase system, he considered it a mistake to standardize too rigidly. There should be room to develop independent approaches to different problems. He stated that he would like to hear in this discussion period from anyone who had deviated from the standard procedures.

He wished to consider first the identification tests, i.e. techniques for the examination of samples, water content tests, grain-size tests and Atterberg Limit tests.

Dr. McLeod reported that his early experience had showed him different values of liquid limit could be obtained from the same soil depending upon whether it was air dried, oven dried or in its natural state. These differences were quite significant. As a matter of course now, he insisted that all liquid limits be conducted on soil at or near their natural water content.

Mr. Peterson agreed with Dr. McLeod that significant variations can result by changing the state of the soil. However, in dealing with a large number of samples, the P.F.R.A. had found the air dried method the best for production.

Mr. Eden introduced the "one-point" method of determining the liquid limit. This method had been originally proposed by the U.S. Corps of Engineers*. By this method, soils of the same geological origin would have a constant slope to their flow line. Hence the liquid limit could be determined by one point through use of the following equation:

$$L.L. = Wn \left[\frac{N}{25}\right]^{\text{Tan } B}$$

where N = No. of blows
Wn = Water content at N Blows
Tan B = Slope of flow line

More recently Olmstead and Johnston** suggested that the flow line had a constant slope for all soils and the liquid limit could be expressed by the equation:

$$L.L. = \frac{Wn}{1.419 - 0.3 \text{ Log } N}$$

For any one number of blows, the two equations are reciprocals.

The U.S. Corp of Engineers had analysed 767 alluvial

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* "Simplification of the Liquid Limit Test Procedure". Technical Memorandum No. 3-286, Waterways Experiment Station, June, 1948.

** Olmstead F.R. and Johnston C.M. "A Comparison of Rapid Methods for Determination of Liquid Limit of Soils". Bulletin 95, Highway Research Board, Washington 1955.

and coastal soils in the southern states and Olmstead had reported on 759 American soils. One hundred and fifty Canadian soils had been analysed and the following table presents the results in terms of Tan B.

Soil Type and Location	No. of Tests	Tan B
Leda clay, Ottawa	100	0.100
Dark laminae, varved clay Steep Rock Lake	31	0.140
Light laminae, varved clay Steep Rock Lake	19	0.098
Average for soils	150	0.108
Alluvial and coastal soils - Southern U.S.A. (Reported by Vicksburg Waterways Experiment Station)	767	0.121
Soils from various locations in U.S.A. (Reported by Olmstead)	759	0.135

To show what this means in terms of liquid limit, consider for example a soil to have water content of 50 per cent at 20 blows. If the liquid limit is calculated using the various values of Tan B, the following are obtained.

Tan B	L.L.
.100	48.9
.140	48.4
.098	48.9
.108	48.8
.121	48.6
.135	48.7

These results would suggest that the single point method could be used if the liquid limit is determined solely for classification purposes.

Mr. Ripley stated that while the single point method reduced the amount of work involved in the test, the three point method provided a check for the operator. He asked how many laboratories used the single point method. None indicated that they did.

Compaction and Density Tests

Mr. Ripley suggested that the Conference consider the relative merits of the standard Proctor test with regard to the modified Proctor compaction tests. He would be interested to hear of any correlation results with the miniature compaction test.

Regarding density determinations, he had found no standard method of determining the maximum density with regard to relative density tests. He would welcome comments on methods of determining the maximum density.

Dean Hardy reported that in some instances with granular soil, no optimum moisture content could be obtained with the Proctor test. In such cases better control could be obtained by finding the maximum density and maintaining a certain relative density. This had been the case with the fill at the Kitimat plant site.

Mr. Torchinsky commented that if the standard Proctor test was conducted, according to standard procedures on coarse granular material, errors could result because all particles larger than the No. 4 sieve were screened out. He asked if anyone else had met a similar situation.

Dean Hardy also thought that the standard method was faulty in this regard. He pointed out that the British Road Research Laboratory suggested the test be conducted on material finer than 3/4 inch and got around the difficulty in all but the very coarsest materials.

Dr. McLeod stated that the A.S.T.M. procedures recommended that all particles coarser than the No. 4 sieve be removed for the standard Proctor test. This was justified in cases where the coarser particles were entirely surrounded by finer particles. But if there was not enough fine material to fill the voids between larger particles, then the method was in error.

Regarding the standard and modified Proctor tests, Dr. McLeod found that generally highway practice favoured the standard Proctor test while in airports where heavier wheel loads had to be considered, the modified test was favoured. It had been his experience that on clay subgrades only 95 per cent of Proctor compaction can be maintained over a period of years while with granular soils, 100 - 110 per cent Proctor density can be maintained.

Dean Hardy had found that it was not practical to get much better than 97 per cent Proctor density on clayey soils. The only advantage he could see in using the modified test was that in some instances the optimum water content corresponded more closely with field conditions.

Dr. Meyerhof commented that the Proctor test had been used for considerable time. There was not much advantage in using the modified method because it also merely obtained a relationship between density and moisture. Densities greater than 100 per cent standard Proctor could be specified.

Dr. Meyerhof agreed that the method of determining maximum density required a standard. In present practice the relative density varies with the method of determining the maximum density. For instance, Terzaghi advocates the use of the standard Proctor test, while at Imperial College, the maximum density is determined by immersing the sample, then subjecting it to a static load, and then vibrating.

Mr. Ripley found the relative density method of control open to question because of the difficulties in obtaining maximum density. It had been his experience that the maximum density obtained depended upon the amount of work expended to obtain it.

Regarding the compaction tests, it was generally the objective of control tests to maintain a certain density. If 100 per cent Proctor was specified, it is natural for the contractor to consider this the maximum density. In some cases where high densities were required, it would be necessary to specify more than 100 per cent Proctor density. This would have a bad psychological effect, one which could easily be overcome by using the modified test. While on the subject, he was familiar with a method in use in California whereby compaction is considered in terms of ft. - lb. per cubic foot.

Mr. Knight asked what tolerance should be accepted when we specify say 95 per cent Proctor density. One per cent either way usually did not mean much to contractors, and yet in some cases a reduction in density of 1 per cent will mean a large loss in strength.

Mr. Lane replied that on unit price contracts the difficulty could be resolved by specifying a minimum density and not restricting the maximum.

Mr. Soterman reported that this problem was met on the dikes for the Niagara project. The problem was to maintain a minimum value of shear strength. Laboratory tests established a relationship between strength and density, and this density was specified as the minimum.

Mr. Ripley thought that this might be true in certain instances, but in other cases it might be wrong because some soil lost its strength with time.

Dean Hardy felt that the Proctor test was still a pretty good criterion. Proctor had evolved it originally as a method of assuring a reasonable standard of construction. On the average job, say a highway embankment, that is all that can be justified, and strength is not a vital consideration.

The Chairman, Mr. Ripley, remarked that whatever the compaction, it should be uniform and he then gave a detailed example to support his statement.

Permeability Tests

Mr. Ripley stated that there were two main methods of determining permeability in the laboratory; first, the falling head apparatus and secondly, by deriving, or measuring permeability in the consolidometer. He considered that a serious limitation on accuracy was placed on the falling head parameter through the use of de-aired distilled water.

Mr. Peterson reported that P.F.R.A. preferred the field test to the laboratory test and most of their determinations were conducted in the field. The laboratory methods not only had inherent errors such as those mentioned by Mr. Ripley, but were dependent upon good undisturbed samples which were difficult to obtain. For accurate determinations, pumping tests were used. For rough determinations a technique had been developed for seepage tests in boreholes using a barrel of water as a supply.

Mr. Adams reported that the Ontario Hydro had found the reverse situation. Field tests had proven unreliable in glacial till presumably because of sand lenses. Good results had been obtained in the laboratory using a small diameter falling head parameter.

Mr. Penner thought that present day laboratory techniques did not pay sufficient attention to the quality of the water. If distilled water was passed through a sample of clay it would leach out the salts and could change the permeability.

Mr. Ripley stated that at present the biggest concern in laboratory testing was to make sure that there were no air bubbles trapped in the system. The difficulty in leaching of salts through use of distilled water could be overcome by determining permeability by the consolidation test.

In reply to a question whether hydraulically placed sand was more impermeable than sand not hydraulically placed, Mr. Ripley did not think there would be much difference, if the sand was compacted by rolling. Recent experience had shown him that hydraulically placed fills could have extremely variable densities and should be compacted after placement. Mr. Ripley considered that field permeability tests are a subject worthy of detailed consideration and suggested they be considered at a future conference.

Strength and Consolidation Tests - led by P.J. Rivard

Mr. Rivard first considered the unconfined compression test. At P.F.R.A. most unconfined tests were not conducted on capped samples. The purpose of the caps was to prevent failure through the ends of the test specimen. With clay shales, a plaster of paris cap was used which extended about 1/4 inch below the top of the specimens. Using a special mould, the cap was easily placed and assured samples with plane, parallel ends. With highly plastic softer materials a tipping cap was used, which would allow the sample to tip slightly if it had a tendency to do so. This method prevented a buckling failure which sometimes does occur.

P.F.R.A. used both the controlled strain and controlled stress test apparatus. The controlled strain apparatus was preferred because the portion of the stress-strain curve after failure could be obtained. This was not entirely feasible with the hanger loading type of controlled stress apparatus.

With the controlled stress test, increment loads equal to about one-tenth of shear strength were added each minute. If pore pressures were measured, often the time interval between increment loads were much longer than one minute. It was observed that saturated samples showed a decrease in strength with increased testing time. Unsaturated samples showed a decrease up to a certain point, then began to increase in strength. This had promoted investigation of what the P.F.R.A. called the creep strength properties of some soils. If the ratio of compressive strength to applied load were plotted against time to failure, it had been observed that after three months the strength dropped to 50 per cent. This raised some interesting questions, e.g., does this loss of strength apply to all materials and what are the factors which enter into the problem?

Mr. Peterson remarked that in the creep-strength tests great care was taken to retain the original moisture content of the specimens. The specimen was enclosed in two membranes with silicone grease between the membranes. The specimen was then immersed in water. There had been fairly good evidence that the specimen did not loose or gain any moisture. This did not mean that moisture migration did not take place within the specimen.

Mr. Ripley asked if the creep-strength relationship could be influenced by the depth from which the sample was obtained.

Mr. Peterson replied that thus far only samples from shallow depths had been studied.

Dean Hardy asked if there had been any field evidence to support the results indicated by the laboratory tests. Mr. Peterson replied that some foundation failures under embankments had occurred with an apparent safety factor of three or greater.

In answer to Dr. McLeod, Mr. Rivard could advance no reason why unsaturated samples at first decreased in strength then increased in strength with time.

Mr. Lucas remarked that similar reductions occurred in the strength of concrete cylinders at very slow rates of testing.

Mr. Baracos wondered if the reduction in strength was a function of time and not of strain.

In answer to an inquiry as to whether oxidation could be the cause of strength reduction, Mr. Rivard replied that care was taken to prevent oxidation in the tests and he did not consider it a factor.

Mr. Lea remarked that at the Third International Conference at Zurich considerable discussion was devoted to creep. Harvard University had been conducting research on the creep of various materials and did not yet have the answer. He had observed one instance of decrease in strength with time, and found the implications of creep very disturbing to the designer. Dr. Skempton, in studies of highly consolidated British clays, had found an explanation for landslides in certain cuts after being stable for many years. He attributed the reduction in strength to localized swelling. This might be true in certain instances, but Mr. Lea did not think that all cases could be explained by swelling.

Dean Hardy reported that along the Alaska Highway there were several failures in clay shales which developed 10 to 12 years after construction. This had been attributed to the gradual disintegration of the shales.

Mr. Adams asked if consolidation might not be a factor. There may be some apparent cohesion due to pore pressures which disappear with time.

Mr. Rivard stated that no drainage was allowed during the test and hence theoretically there could be no consolidation. This did not mean that there was no migration of water within the test specimen.

Next Mr. Rivard dealt with the triaxial test and pore pressure measurements. In long term triaxial tests, piston friction became an important factor. In order to reduce the friction, Harvard University had developed a small diameter piston with ball bushings. Decrease in fluid pressure by leakage at the bushing was prevented by a special grease seal.

Mr. Rivard considered there were two approaches to the measurement of pore pressures. One method, which was developed by M.I.T. was to introduce a porous pilot head into the sample near the failure plane. The other method was to measure pore pressures through use of saturated porous plates at the top or bottom of the test specimen. Exponents of the pilot method claimed the porous plate method did not measure the pore pressure where they should be measured, i.e. near the plane of failure. The pilot method, on the other hand, introduces a foreign object to the specimen and hence may not result in a true test.

There also seemed to be two schools of thought regarding the conduct of the test. One method was to establish a meniscus level before the test and prevent it from moving by introducing or releasing air pressure from the measuring apparatus. The other school considered it impossible to exclude all the air from the measuring system and allows the meniscus to fluctuate. One school maintains that if movement of the meniscus is allowed, then drainage takes place, while the other does not consider this a serious factor.

A noticeable time lag had been noticed in pore pressure response. In porous materials, the response was almost instantaneous, while in other tests on plastic materials, times up to three hours were required for the pore pressure to develop completely.

Mr. Torchinsky stated that one method of eliminating piston friction in the triaxial test was to mount the proving ring inside the test chamber. This method was not widely used and he asked if anyone had experience with it.

Dr. Meyerhof reported that one method of speeding the response of pore pressures was to wrap the specimen with a layer of filter paper. This method was used in England and did reduce the time lag.

Mr. Baracos reported he had difficulty making pore pressure measurements in some soils because a seal developed at the ends of the specimen and did not allow the true pressures to be measured.

Mr. Rivard stated that the pore pressure measurements were developed so that effective stresses could be ascertained from the quick test, eliminating the need for time-consuming slow tests. However, it had been found that a condition of prestress was being introduced which gave the samples added strength. Thus, pore pressure measurements were not the complete answer.

In discussing the consolidation test, Mr. Rivard reported difficulties with electrochemical action involving the soil and brass rings. In some cases, this action had hardened the samples. This action was noticed only in long-term tests. To overcome the difficulty, the test cell and accessories were constructed entirely of plastic. While it solved the problem of electrochemical action, it had introduced some difficulties in the calibration of the apparatus for inherent compression.

Mr. Torchinsky, in dealing with swelling soils, considered the regular consolidation of not much significance. As soon as water is added, the sample swells or exerts a swelling pressure, thus the P-e curve could not be used to predict deformation conditions directly. At the University of Saskatchewan, they were endeavouring to conduct consolidation tests without the use of water.

Dean Hardy stated that more and more problems require detailed studies of the magnitude of settlements. In his experience he had found cases where on one hand, settlements up to 4 feet were anticipated and accommodated, and, on the other hand, the deformation could not exceed 15/1000 inch.

Another problem which required much research was that of foundations for vibratory loads. Early work conducted in Germany indicated that the soil had a natural frequency. This was erroneous since not only the soil type but the mass of soil determined the resonant frequency. He had found Tschebotarioff's approach to the problems of vibrations the most realistic.

Mr. Neilson asked for more details as to why the early German work on soil vibrations was considered unreliable.

Dean Hardy replied that the German determinations of natural frequency of soil were found using a certain machine with a base of 10 square meters. This system would have a resonant frequency, but the resonant frequency determined was in no way related to that of a larger system on the same soil. This was true because an undetermined mass of soil is affected by the foundation. Tschebotarioff indicates that as the bearing area is increased, the resonant frequency is decreased.

Resonant frequencies could be quite serious. For example, one installation with which he was familiar, had operated successfully for a number of months at 60 per cent of total power. When the power output was stepped up to 80 per cent failures began to occur within a few hours.

Mr. Ripley then brought up the subject of sensitive clays. There were several things he would like to know. Can the consolidation test, for instance, be used to predict settlements with any accuracy, and how much load can be applied without breaking down the soil structure?

Mr. Peckover reported that for construction of the St. Lawrence Seaway answers for such questions must be found for the Leda clay of Eastern Canada. He then outlined the large-scale test program initiated by the various agencies engaged on the project.

Mr. Bazett reported that the shear strength obtained from the full scale field test matched the shear strength obtained from vane tests, following Skempton's method of analysis. Mr. Bazett also commented that Bjerrum in Norway had recently reported that sensitive clays can be treated as normal clays but that special techniques in obtaining samples must be followed.

Mr. Ripley in summing up the discussion stated that he had found it personally very interesting and stimulating. Because the field was so large it was not possible to cover all the points. The discussion had shown there had been a good many points of view in the approach to laboratory testing of soils but he considered this was a good sign because of the many problems peculiar to each type of soil.

Section 6

General Business

(1) Soil Classification

The Chairman, Mr. Legget, stated that last year the Conference as a whole had considered the matter of the "Suggested Standard on the Identification and Description of Soils". As a result of the discussion at last year's conference a special subcommittee under the chairmanship of Mr. R. Peterson had been set up. A draft document embodying the field description of soils had been prepared by Mr. Eden and submitted to the Subcommittee for comment in November. This special Subcommittee met the evening of December 16th and the Chairman asked Mr. Peterson if he would report on the meeting.

Mr. Peterson reported that he, as chairman of the Subcommittee, was anxious to get some action on the matter. The Subcommittee meeting showed that there was a considerable variance in the opinion of the Subcommittee members. Some considered that the draft, "Suggested Standards for the Field Identification of Soils for Engineering Purposes", did not go far enough while others thought it was much too complicated for the field man. The Subcommittee decided that the greatest need was a short document which could be used by the field man and that it be restricted to a descriptive system for soils and a definition of terms. This descriptive system should conform with the unified system in use in the United States and the British Code of Practice, if possible. It would be the basis from which expansion could follow for a more detailed soil classification system. The title of the new document would be, "Suggested Field Description of Soils for Engineers".

Mr. Peterson then outlined how the Subcommittee would work toward this document. All the subcommittee members had been asked to go over the document prepared by Mr. Eden in detail and submit their comments to him. Mr. Peterson proposed to prepare the new document, "Suggested Field Description of Soils for Engineers", and re-submit it to the Associate Committee. If the committee approved, this document would be circulated widely and Mr. Peterson hoped that a satisfactory document would be available for the next Soil Mechanics Conference.

(2) Canadian Section of the International Society on Soil Mechanics and Foundation Engineering

The Chairman reported that Mr. Schriever, the Secretary of the Canadian Section, had been unable to attend today's session, but that he had prepared a report and asked that it be presented by Mr. Eden.

At last year's Canadian Soil Mechanics Conference some time was devoted to the International Society mainly in connection with the Third International Conference on Soil Mechanics and Foundation Engineering held in Zurich, Switzerland in August, 1953. Four members of the Canadian Section who attended this Conference reported on the technical sessions and the decisions of the Executive Committee. Since then a number of matters have been dealt with by correspondence mainly with Mr. Banister in London, England, the new secretary of the International Society. Members of the Canadian Section have been informed about the more important matters by circular letters which were sent out during the year. The purpose of this report is to summarize these developments briefly.

Mr. Banister was informed by letter of some suggestions on how the International Conference might be improved in the future to be of greater benefit to those attending the Conference. The main points were that more time should be allowed for actual discussion by cutting out consecutive translation, that the quality of the discussions should be improved and that smaller discussion groups should be formed for the last one or two days of the Conference. Mr. Banister welcomed these comments and placed them before the British Organizing Committee for the next Conference.

Early in the year a membership list of the Canadian Section was sent to Mr. Banister. At the present time there are approximately 175 members. Not all of these are active as can be seen from the fact that only 33 annual reports were received on our request earlier in the year. No membership fee is charged to the members as it is paid by the Associate Committee on Soil and Snow Mechanics.

It may be recalled that the Swedish Geotechnical Institute proposed at the Zurich Conference to organize an International Soil Mechanics Literature Service, which would consist of maintaining a record of abstracts of soil mechanics literature published all over the world in the form of a card index and to make copies of these cards available to engineers, laboratories and societies for a subscription fee. The results of the questionnaire which were sent out by the Swedish group to survey all countries with regard to the prospective number of subscribers was not encouraging and did not reach a number considered sufficient to run the literature service on a self-sustaining basis. Reasons for the lack of greater interest were suggested by Professor Casagrande and considerable correspondence has since taken place between him and Mr. Kjellman of the Swedish Geotechnical Institute. This correspondence covers mainly the classification system to be used, the scope and the name "Geotechnics".

The Chairman stated that he personally thought that one reason for the poor response to the Swedish card index service was that they failed to use the Universal

Decimal Classification system of classification which was being adopted almost universally. Mr. Lea shared that view and considered another reason for the poor response was that the cards circulated from Sweden did not ask definite questions which would require a decision. Thus Mr. Lea thought that many organizations had to put off the decision but were actually considering it.

Dr. McLeod suggested that the name "Geotechnics", as proposed by Kjellman was perhaps not understood by many engineers. Dr. McLeod would personally prefer the term "Geotechnology". Mr. Lea suggested that "Soil Mechanics" as suggested by Casagrande was too well entrenched to be changed at present. He suggested that the American view be considered, that is abbreviating Soil Mechanics and Foundation Engineering into Soil Engineering. The Chairman stated that he would express these views to Mr. Kjellman.

The six Canadian papers which were presented at the Zurich Conference have been published by the Associate Committee on Soil and Snow Mechanics as a separate publication (Technical Memorandum No. 30). This publication can be obtained on request from the Secretary of the Associate Committee.

The Chairman reported that the Fourth International Conference would be held in Great Britain in September, 1957. Papers would have to be submitted to the Conference well in advance in order that the Proceedings may be prepared for discussion at the Conference. He asked that the members of the Canadian Section consider the matter of Canadian contributions and have definite proposals for next year's Canadian Soil Mechanics Conference.

Based on the Annual Reports received from individual members an annual report of the Canadian Section was prepared covering the period of June, 1953 to June, 1954. This report was sent to Mr. Banister and also to all members. In due time an annual bulletin of the International Society will be received containing summaries of annual reports of all member countries.

During the summer members living in the Montreal, Ottawa and Toronto areas had the pleasure of hearing Dr. Skempton, who is Regional Vice-President of the International Society for Europe, and Dr. A. von Moos, who was secretary for the Zurich Conference, address the local soil mechanics group.

In a recent circular letter the new statutes of the International Society were sent to all members. It was also mentioned that a small dictionary of technical terms used in soil mechanics and foundation engineering (in six languages; English, French, German, Swedish, Portugese and Spanish) had been prepared by Dr. von Moos and his staff and that this book was available at a price of approximately $1.75 from Switzerland. Dr. von Moos also informed us that there are still a number of copies of the proceedings of the Zurich Conference available.

(3) Regional Reports

Unfortunately, there was no one from the Maritime Provinces present at the Conference at this time. The Chairman stated that a Maritime Regional Conference had been held in April in Fredericton, New Brunswick. Dr. McLeod, Dean Hardy, and Mr. A.W. Johnson of the Highway Research Board were guest speakers. The meeting was attended by over 120 engineers and he considered it a very successful conference.

Montreal Group: Mr. J.C. Brodeur, Chairman of the Montreal group was not in attendance but submitted a report of his group in a letter.

The following is a summary of luncheon meetings held by the local soil mechanics group in Montreal during the past year:

Date	Speaker	Subject
Jan. 29/54	Dr. G.G. Meyerhof	"Recent Study of Foundation Behaviour and its Application to Design".
March 4/54	Mr. G.D. Campbell	"Highway Soils Engineering".
May 19/54	Mr. Geo F. Flay	"A Unique Type of Foundation".
June 1/54	Dr. A.W. Skempton	"Practical Application of Soil Mechanics to Civil Engineering".

July 5/54	Dr. A. von Moos	"Pleistocene Geology".
Nov. 9/54	Mr. W.H. Peterson	"The Toronto Subway - Soil Conditions and Foundation Problems".
Dec. 9/54	Dr. N.W. McLeod	"Some Applications of Soil Mechanics to Airport and Highway Design".

At the first fall meeting Mr. R. Quintal was elected Chairman of the Montreal soil mechanics group for the coming year.

Ottawa Group: Reported by Mr. W.W. Gruber

Mr. Gruber reported that the Ottawa group had held seven evening discussion meetings as well as an organizational meeting and a field demonstration of sampling equipment since the last Canadian Soil Mechanics Conference. The topics discussed are listed below:

1. Frost Action - E. Penner, J. Sutherland and E.B. Wilkins.
2. Slope Stability - G.C. McRostie , C.B. Crawford and W.J. Eden.
3. Corrosion of Metals in Soils - T.R.B. Watson.
4. Maryland and Washo Test Roads - A.W. Johnson.
5. Shear Strength of Saturated Clays - Dr. A.W. Skempton.
6. Geotechnical Problems in Switzerland - Dr. A. von Moos.
7. Program for Frost Action Studies- E. Penner.

Toronto Group: Reported by Mr. J. A. Knight

Mr. Knight reported that during the year eight meetings had been held with an average attendance of about 35 persons. The mailing list of the Toronto group now numbered 110. The subjects were varied and attendance appeared to depend on the subject to be covered and the speaker. Mr. Knight stated that the Toronto group was very pleased to have Mr. N.D. Lea, past Chairman of the Montreal group as a member.

Prairie Provinces: Reported by Dean A.E. Macdonald

Dean Macdonald stated that this year he could just report for the Provinces of Manitoba and Saskatchewan since he had been unable to contact Dean Hardy regarding activities in Alberta but this did not necessarily mean that there were no activities in Alberta. In Saskatchewan a concrete research group had met in Saskatoon in November and the old problem of soil sulphate action on concrete had come up for considerable attention at the meeting. In Manitoba there was no organized group such as in Toronto, Montreal, and Ottawa, but that some meetings of the Civil Section of the Engineering Institute, Winnipeg Branch, had been devoted to soils. At the University of Manitoba an active soils laboratory was in operation under the direction of Professor A. Baracos. Professor Baracos, under the sponsorship of the National Research Council, was working on an experimental slab house foundation and on vertical ground movements. Under the sponsorship of Prairie Road Builders, studies on soil compaction had been undertaken. Dean MacDonald also reported he had received a letter from Mr. George Baldry, an underpinning contractor, in which he reported the loss of a workman's life in an open well from gas poisoning.

Vancouver Group: Reported by C.F. Ripley

Mr. Ripley reported that there was increased activity in Vancouver and that an organizational meeting had been held last winter to determine the interest and responsibility for the formation of a local group. Agricultural scientists and engineers had been invited to a meeting at which Mr. R.F. Legget was guest speaker. The meeting showed that the Vancouver group preferred very informal organization for the time being. Three meetings had been held since the organizational meeting. They took the form of visits to a commercial soil mechanics laboratory and a university soil mechanics laboratory. Mr. Ripley thought that in spite of the loose organization the interest was definitely there and he looked forward to increased attendance.

(4) <u>Subcommittee on Canadian Landslides</u> - Reported by
Mr. N.D. Lea

Mr. Lea reported that the second meeting of the Subcommittee on Canadian Landslides was held on the evening of December 16th. It was really the first meeting in which all regions were represented on the Subcommittee. The Subcommittee at present consists of:

N. Gadd - Geological Survey of Canada.
J.C. Chagnon - Quebec Streams Commission.
Prof. J. Hurtubise - Ècole Polytechnique, Montreal.
G.C. McRostie - Consulting Engineer, Ottawa.
W. Trow - H.E.P.C. Toronto.
Prof. A. Baracos - University of Manitoba, Winnipeg.
Mr. R. Peterson - P.F.R.A., Saskatoon.
Dean R.M. Hardy - University of Alberta, Edmonton.
C.F. Ripley - Consulting Engineer, Vancouver.
N.D. Lea - Foundation Co. of Canada, Toronto (Chairman).
M. Bozozuk - N.R.C. Ottawa,(Secretary).

In addition, several guests were in attendance. The terms of reference of the Subcommittee were decided and these would be submitted to the next meeting of the Associate Committee on Soil and Snow Mechanics. Generally, the terms of reference were as follows:

(1) To co-ordinate research on Canadian landslides;
(2) To isolate specific problems of Canadian interests on landslides and to promote research on these problems.

One of the main functions of the Subcommittee would be that of gathering and becoming familiar with the literature dealing with Canadian landslides. The library of the Division of Building Research, National Research Council, would keep a record of all published material but there was no means of obtaining reference to private reports and such items which might be reported in local newspapers. Mr. Lea asked if every member of the Conference would draw to the attention of the Subcommittee any such reports with which he may be familiar.

(5) The Directory of Facilities Available in Canada for Obtaining and the Testing of Soil Samples

The Chairman mentioned that at last year's Conference it had been decided to compile a directory of drilling contractors with facilities for obtaining undisturbed soil samples and also listing laboratories which had facilities to conduct engineering tests on soil samples. A questionnaire had been prepared to gather information for the directory. The technical press had carried a press release recently advising interested parties of the questionnaire. The Chairman asked if anyone doing this work or knowing of anyone else who should be listed in the directory to ask them to request a copy of the questionnaire from the Secretary of the Associate Committee on Soil and Snow Mechanics.

(6) The Ninth Canadian Soil Mechanics Conference

The Chairman reported that it was the usual pattern to have two out of three conferences in Ottawa with the third at some outside point. The 7th and 8th Conferences had been held in Ottawa which would mean the 9th meeting would be held elsewhere. Two invitations had been received one from Saskatoon and one from Vancouver, through the University of British Columbia. The decision as to where the next meeting would be held would be decided by the Associate Committee which would meet either in February or March.

(7) Publications

The Chairman, in answering a query from Mr. Knight, reported that there was a definite need for a journal which would publish short technical papers dealing with topics which normally would not be carried by such publications as the Engineering Journal. The Canadian Journal of Technology was one possibility but it carried such a variety of diversified subjects that it was not too well known. At present consideration is being given to changing the name of the Canadian Journal of Technology, to the Canadian Journal of Engineering Research and that, if such a change was made, the future contents of this journal would be devoted exclusively to engineering topics. This would be a logical outlet for Canadian papers on specific soil problems.

APPENDIX A

List of those Present at the Eighth Annual Canadian Soil Mechanics Conference

Adams, J.F., Hydro-Electric Power Commission, St. Lawrence Project, Cornwall, Ont.

Andersen, V., Aluminum Co. of Canada Ltd., Montreal, Que.

Armstrong, E.F., Dept. of Transport, Montreal, Que.

Baracos, A., University of Manitoba, Winnipeg, Man.

Bazett, D.J., Hydro-Electric Power Commission, Toronto, Ont.

Beauchemin, R.O., Canada Cement Co., Montreal, Que.

Beaudry, A., City of Montreal, Montreal, Que.

Bilodeau, P.M., Quebec Dept. of Highways, Quebec, Que.

Binks, W.R., Dept. of Public Works, Trans Canada Highway Division, Ottawa, Ont.

Bonn, W.E., Consulting Engineer, 51 Harper Ave., Toronto, Ont.

Bourget, S.J., Dept. of Agriculture, Ottawa, Ont.

Braun, J.F., Aluminum Company of Canada Ltd., Montreal, Que.

Bray, J., Racey-MacCallum & Associates, Montreal, Que.

Brown, P., Testing Laboratories, Dept. of Public Works, Ottawa,Ont.

Brown, W.C.E., Forestry Branch, 238 Sparks St., Ottawa, Ont.

Brownridge, F.C., Ontario Dept. of Highways, Toronto, Ont.

Carmichael, J.W., Dept. of Public Works, Ottawa, Ont.

Casey, J.J., Ontario Dept. of Highways, Toronto, Ont.

Chlipalski, A., Dept. of National Defence, Ottawa, Ont.

Clark, L.E., Brunner Mond Co., 137 Wellington St., W., Toronto, Ont.

Coates, D.F., McGill University, Montreal, Que.

Cooper, D.J., Central Experimental Farm, Ottawa, Ont.

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

Davis, M.M., Ontario Dept. of Highways, Toronto, Ont.

Delisle, M., Consulting Engineer, 10355 Hogue St., Montreal, Que.

Devitt, Major H.E.A., Chief Engineer's Office, Army Headquarters, Ottawa, Ont.

Dufour, M., National Borings & Sounding, Canada Cement Building, Montreal, Que.

Dujay, W.C., Dept. of Transport, Ottawa, Ont.

Dutton, V.L., Dept. of Transport, Ottawa, Ont.

Eckel, E.B., U.S. Geological Survey, Denver, Colorado.

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

Farantatos, G., Ontario Dept. of Highways, Toronto, Ont.

Farrar, J.L., Forestry Branch, Dept. of Northern Affairs and National Resources, Ottawa, Ont.

Foures, G.H., Dept. of Public Works, Ottawa, Ont.

Fournier, R., Quebec Provincial Government, Quebec, Que.

Fowler, E.L., Materials Testing Laboratories Ltd., Edmonton, Alta.

Gadd, N.R., Geological Survey of Canada, Ottawa, Ont.

Gardner, H.C., Canadian Longyear Ltd., North Bay, Ont.

Gorman, G.A., Hydro-Electric Power Commission, Box 299, Niagara Falls, Ont.

Graves, A.H., Dept. of Public Works, Ottawa, Ont.

Greenberg, M., Dept. of Transport, Winnipeg, Man.

Gruber, W.W., Canal Services, Dept. of Transport, Ottawa, Ont.

Hall, Per, Foundation of Canada Engineering Corp. Ltd., Montreal, Que.

Hardy, R.M., University of Alberta, Edmonton, Alta.

Harper, A.E., E.J. Longyear Co., Minneapolis, Minn., U.S.A.

Helwig, C., University of Toronto, Toronto, Ont.

Henderson, E., Dept. of Mines and Technical Surveys, Ottawa, Ont.

Hipperson, E.P., Construction Borings Ltd., Montreal, Que.

Hunter, L., Dept. of Transport, Moncton, N.B.

Hurtubise, J.E., Ecole Polytechnique, Montreal, Que.

Jones, F.W., Dept. of National Defence (DVD), Ottawa, Ont.

Kalbfleisch, W., Dept. of Agriculture, Ottawa, Ont.

Kempton, D.R., Hydro-Electric Power Commission, Toronto, Ont.

Kennedy, M., Racey-MacCallum & Associates, Montreal, Que.

Knight, J.A., Brunner Mond Co., 137 Wellington St.,W., Toronto, Ont.

Kofmel, K.E., Dept. of National Defence (DED), Ottawa, Ont.

Lane, D.A., Dept. of Transport, Ottawa, Ont.

Lea, N.D., Foundation Company of Canada, Toronto, Ont.

Lee, H.A., Geological Survey of Canada, Ottawa, Ont.

Lucas, J.W., Dept. of Public Works, Testing Laboratories, Ottawa,Ont.

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

MacHattie, L.B., Dept. of Northern Affairs and National Resources,
Forestry Branch, Ottawa, Ont.

MacIver, S.M., National Harbours Board, Ottawa, Ont.

McLeod, N.W., Imperial Oil Ltd., Toronto, Ont.

McRostie, G.C., Consulting Engineer, Ottawa, Ont.

Martin, O., Queen's University, Kingston, Ont.

Matich, M.A.J., Geocon Ltd., Toronto, Ont.

Merrill, C.L., Dept. of Northern Affairs and National Resources,
Ottawa, Ont.

Meyerhof, G.G., Foundation Company of Canada, Montreal, Que.

Millette, J.F.C., New Brunswick Soil Survey, University of
New Brunswick, Fredericton, N.B.

Neilson, A.O.H., Aluminum Company of Canada Ltd., Montreal, Que.

Nelson, A.R., Raymond Concrete Pile Co. Ltd., Montreal, Que.

Nelson, J.C., Spencer, White and Prentis of Canada Ltd.,Toronto,Ont.

Nurse, W.G., Dept. of Transport, Montreal, Que.

Ostrum, J.A., Dept. of Transport, Toronto, Ont.

Paskevicius, A., Dept. of National Defence, RCAF/AMTS/DCED,
Ottawa, Ont.

Pastien, W.J., Dept. of Public Works, Testing Laboratories,
Ottawa, Ont.

Paterson, J.D., Frazer Duntile Co., Ottawa, Ont.

Patterson, F.W., H.G.Acres & Co. Ltd., Niagara Falls, Ont.

Peckover, F.L., St. Lawrence Seaway Project, 685 Cathcart St.,
Montreal, Que.

Pepin, R., Quebec Highway Laboratory, Quebec, Que.

Perkins, C.L., Imperial Oil Ltd., Toronto, Ont.

Peterson, R., Prairie Farm Rehabilitation Administration,
Saskatoon, Sask.

Piette, G., Consulting Engineer, 13 Place d'Aiguillon, Quebec, Que.

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

Pugh, W.L., Aluminum Company of Canada Ltd., Montreal, Que.

Radforth, N.W., McMaster University, London, Ont.

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

Ripley, P.O., Dept. of Agriculture, Ottawa, Ont.

Rivard, P.J., Prairie Farm Rehabilitation Administration,
Saskatoon, Sask.

Robinson, J.M., Dept. of Northern Affairs and National Resources,
Forestry Branch, Ottawa, Ont.

Rubenbauer, J.M., Dept. of Public Works, Testing Laboratories,
Ottawa, Ont.

Rutka, A., Dept. of Highways, Toronto, Ont.

Shaw, J.F., Dept. of National Defence (RCAF), Ottawa, Ont.

Shechter, A., Dept. of Public Works, Ottawa, Ont.

Soterman, L.G., Hydro-Electric Power Commission, Box 999,
Cornwall, Ont.

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

Sully, G.B., Dept. of Northern Affairs and National Resources, Forestry Branch, Ottawa, Ont.

Sutherland, J., Dept. of Transport, Ottawa, Ont.

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

Tubbesing, K., Racey-MacCallum & Associates, Montreal, Que.

Watt, J.S., National Harbours Board, Ottawa, Ont.

Weichel, A.E., Dept. of Transport, Toronto, Ont.

Wilson, B.E., Spencer, White and Prentis of Canada Ltd., Toronto, Ont.

Wright, D.T., Queen's University, Kingston, Ont.

Division of Building Research, N.R.C.

Legget, R.F. Director

Bozozuk, M.

Brown, R.J.E.

Burn, K.N.

Crawford, C.B.

Eden, W.J.

Guibord, J.A.O.

Johnston, G.H.

MacFarlane, I.C.

Penner, E.