Proceedings of the Seventh Canadian Soil Mechanics Conference: December 10 and 11, 1953

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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

PROCEEDINGS OF THE SEVENTH CANADIAN SOIL MECHANICS CONFERENCE

DECEMBER 10 AND 11, 1953

TECHNICAL MEMORANDUM NO. 33

Ottawa
September 1954

FOREWORD

This is a record of the Seventh Annual Conference of active Canadian workers in the field of soil mechanics which was held in Ottawa on December 10 and 11, 1953. 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 10 was devoted to a discussion of the Third International Conference on Soil Mechanics and Foundation Engineering held in Switzerland, August, 1953. Mr. R.F. Legget acted as chairman, and discussion was lead by Messrs. W.R. Schriever, N.D. Lea, G.C. McRostie, and Dr. N.W. McLeod. On the afternoon of December 10 a suggested Standard on the Identification and Description of Soils for Engineering Purposes was discussed, led by Mr. R.F. Legget, followed by a brief business meeting. Short papers made up the program for the morning of December 11 with Dean A.E. Macdonald and Dean R.M. Hardy acting as chairmen. Other short papers were included in the afternoon program with Mr. R. Peterson and Mr. C.B. Crawford as chairmen.

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SESSION OF DECEMBER 10, 1953

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. For the first time it was possible to meet in the building of the Division of Building Research, which had been dedicated by the Rt. Hon. C.D. Howe on October 23, 1953. He invited anyone who was interested to tour the building and see work in fields other than soil mechanics that was in process.

He regretted that Professor I.F. Morrison of the University of Alberta was unable to attend because of illness.

In announcing the program for the day, the attention of the conference was invited to an evening meeting at which slides of various features of the Third International Conference would be shown. The evening meeting was not part of the formal program of the conference, and would be a very informal gathering.

At the chairman's request, Mr. W.R. Schriever, secretary of the Canadian Section of the International Society read a letter of greeting received from Mr. A. Cummings, Vice-President for North America of the International Society.

Discussion of the Third International Conference on Soil Mechanics and Foundation Engineering

led by W.R. Schriever, Dr. N.W. McLeod, N.D. Lea, and G.C. McRostie.

(a) Executive Committee Meetings of International Society

Mr. Schriever and Dr. McLeod had acted as Canadian delegates at the executive committee meetings of the International Society. The following summary covers the items discussed.

(i) The Statutes of the International Society were considerably revised;

(ii) The finances of the International Society were in good shape; U.N.E.S.C.O. had granted $500 to the Society;

(iii) The official languages were to remain English and French;

(iv) Membership of the Executive Committee was limited to National Societies as before and there would be no change in membership fee;

(v) Five vice-presidents had been appointed with a view to facilitating "regional" conferences, viz., Skempton for Europe, Cummings for North America, Vargas for South America, Hanna for Africa and Hoshino for Asia;

(vi) Professor Taylor of the U.S.A. was succeeded by Mr. Banister of Great Britain as Secretary of the International Society;

(vii) Annual Reports of the National Societies had been found difficult to distribute especially in countries with large membership; the Secretary will undertake the preparation of an annual bulletin which will summarize all the annual reports and distribute it to all members;

(viii) The next International Conference will be held in Great Britain either in 1957 or 1958;

(ix) During the Conference a subcommittee studied the subdivision of the field of soil mechanics for the Conference's technical sessions as there had been several complaints regarding the proper subdivision for the papers; it was decided to omit the subdivision from the Statutes and leave the question to the Organizing Committee of the next Conference;

(x) Before the Third Conference, the Swedish National Committee and the Royal Swedish Geotechnical Institute had circulated a memorandum proposing that the International Society institute a card index and abstract service of soil mechanics literature, together with a new system of classifying soil mechanics literature; a special subcommittee set up to study this matter, requested the Royal Swedish Geotechnical Institute to administer the abstracting service with each National Society informing the Royal Swedish Geotechnical Institute of each nation's contributions. The question of literature classification system was still under study; proposals had been made to modify the sections of the universal decimal classification system on soil mechanics.

Following Mr. Schriever's report on the executive committee meetings, Dr. McLeod and Messrs. Lea and McRostie gave accounts of the technical sessions in the following order:

(b) Report on Session 1 (Theories and Hypothesis of General Character)

Session 2 (Laboratory investigations)

Session 6 (Road and Runway Construction)

(c) Report on Session 3 (Field investigations)

Session 7 (Earth Pressure)

Visit to Royal Swedish Geotechnical Institute, Stockholm, Sweden

(A note by Mr. Lea on his visit to the Royal Swedish Geotechnical Institute is included as Appendix B.)

(d) Report on Session 4 (Foundations of buildings and dams)

Session 8 (Stability of dams and slopes)

General Discussion on Third International Conference

Dr. McLeod reported that in the keynote address at the Conference, Dr. Terzaghi had deplored the strictly theoretical approach to soil mechanics. Since there were so many variables, an empirical approach must be used in practice. Hansen of Denmark, took the opposite view at the Conference. Dr. McLeod could not agree with Terzaghi, because such a viewpoint needs a great deal of experience for background. In the case of young engineers, it was necessary to have a fundamental background, otherwise the empirical approach could lead to serious mistakes.

Mr. Legget asked Dr. McLeod if there was much discussion on the bearing capacity of flexible pavements. Dr. McLeod reported that although the U.S. Corps of Engineers had not presented a paper on their method, they had taken issue with several points in his paper. They had criticized his approach to the problem of multiple wheel assemblies on heavy aircraft. Dr. McLeod stated that the chief difference between his method and that of the U.S. Corps of Engineers was the use of the soaked C.B.R. value. Mr. Peterson mentioned that at a recent A.S.C.E. meeting there had been a discussion on this point. It was suggested that the difference in traffic intensities between American and Canadian conditions may be a cause of the difference in design approach. Dr. McLeod disagreed with this point as traffic intensities had been considered in his design approach.

Mr. McFarlane asked if anyone could give a résumé of the current thinking on the measurement of shear strength. In reply, Dr. McLeod suggested that there seemed to be three schools of thought viz: Casagrande in the United States, Geuze in the Netherlands, and Skempton in Britain. Dr. Casagrande considered the rate of load application to be very important. Casagrande used the triaxial test to determine values of cohesion and internal friction. In the Netherlands, Dr. Geuze preferred a rheological approach in which the rate of strain is more important than the actual method of measuring shear strength. Dr. Skempton, on the other hand, believes that the approach outlined by Hvorslev in 1936 could be safely applied for the determination of shear strength. Dr. McLeod stated that there had been three papers on direct shear tests presented at the Conference.

Mr. Bozozuk asked how the Dutch cell test compared with the triaxial test. Dr. McLeod replied that where the triaxial test requires several test specimens in order to obtain the Mohr envelope, the cell test requires only one specimen. The cell test was developed in the Netherlands and to date its use has been confined to that country.

Dr. Radforth asked if there had been any discussion on the transpiration losses of different types of plants, and if there was a satisfactory method of dealing with the problem. Dr. McLeod replied that the problem was recognized with regard to movements in clays. It was also recognized that different types of trees had different rates of transpiration.

Regarding highway research, Dr. McLeod thought that there would be some valuable contributions from Germany in the future. Highway research, which had been disrupted by the wars, was becoming active once again in Germany.

On the conference in general, Mr. Schriever reported that the practice of making consecutive translations into the two languages took up a great deal of time and hence curtailed the discussion. Mr. Lea stated that the manner in which the discussion was conducted left much to be desired as half the discussion period had been taken up by papers which were not on the program. He suggested that the Canadian Section put forward written suggestions to the International Society regarding the conduct of discussions at future conferences.

Section 3

Discussion on Suggested Standards for the Identification and Description of Soils for Engineering Purposes.

led by R.F. Legget

Before the meeting, everyone who had indicated he would be in attendance had received a draft copy of the document, requested by previous meetings, on the above topic. Since it is not possible to include this document in the proceedings, discussion dealing with specific points has been omitted. In general discussion, Mr. Legget explained that the draft under consideration was the result of suggestions tabled at past conferences. These suggestions had pointed to the need for a document co-ordinating existing information, so that soil description could have a common basis. The document which was distributed attempted nothing new; it was merely a digest of existing publications. Its purpose is fourfold:

(i) to provide a basis for the accurate description of soils in the field;

(ii) to provide a basis for the accurate description of soils in the laboratory;

(iii) to suggest a means by which soil types could be represented on engineering drawings; and

(iv) to select from the many tests available on soils those which suggest the best method of identifying a soil for a particular purpose.

The comments received at this session would be incorporated into a revised draft and again circulated, if reasonable agreement could be obtained on this draft.

Mr. Lea stated that he was of the opinion that a Canadian standard should be in agreement with other well accepted classification systems such as the Unified System used by the U.S. Bureau of Reclamation and the U.S. Army Corps of Engineers. The graphical symbols should parallel the Unified System exactly because there was such a large exchange of information between Canada and the United States.

Mr. Lea thought there was no need for the discussion of laboratory testing. It was necessary only to define terms in relation to the properties determined in the laboratory. Such a standard should be short, if possible, printed on both sides of a single sheet of paper.

Mr. Legget replied that the draft circulated for comment had many explanations in it which would not be included in the final form. It was never intended that it should be a laboratory manual, much of the section on laboratory explanation was devoted to explanation of principles behind the various laboratory criteria used. While the Canadian system should conform closely with the American Unified System, it should conform as well with the British system, since there was also a large exchange of information between Canada and Great Britain.

Mr. Wilkins had discussed the matter of soil classifications with other engineers in the Department of Transport. In general, the Department of Transport would prefer to use a quantitative system based on Atterberg limits, grain size, density and water content rather than qualitative soil descriptions. The Department of Transport field engineers had tried estimating these quantities in the field, and in a short time became proficient. Mr. Wilkins was of the opinion that more accurate field descriptions could be obtained in this manner.

Dean Macdonald was of the opinion that it was undesirable to have a third independent system in use in Canada and that an effort should be made to arrive at a system in Canada which would conform in general with the American and British systems.

Mr. Peterson considered the Unified System an excellent piece of work and that the Canadian system would do well to fit in with it. He noted that the division between sand and gravel should be the No. 4 sieve, as men working both with concrete and soils would use the same limit between sand and gravel. Mr. Peterson regarded the "Activity Chart" as being too premature to be included in such a standard.

Mr. Ripley had not had an opportunity to discuss the draft with other engineers in British Columbia. He reported that the B.C. Highway Department had been seeking to establish a soil classification system, and was considering the Public Roads system. Mr. Ripley thought, however, that the Unified System might well be adopted.

Dr. Radforth considered that the definitions of such words as "identification" and "classification" should be kept in mind in the discussion. He was of the opinion there were quite a number of engineers who used the words without knowing their meaning. This would lead to misunderstanding. Dr. McLeod suggested that the use of "classification" and "identification" was a problem in semantics, and thus did not consider the problem of definitions as being serious.

Dr. Meyerhof thought the difficulty could be resolved by referring to the British system as detailed in the Code of Practice on Site Identifications. Using this system, soil is first described, then identified, and finally classified. This, to him, seemed to be the logical order. Commenting on the adoption of a standard, Dr. Meyerhof considered that the British system and the Unified System agreed closely, and we might well adopt either one. Regarding the inclusion of a section on pedological classification, this might be omitted since this system would be for engineering purposes. Dr. Meyerhof also considered it too early to adopt the activity chart in such a document, as this was primarily a research tool, and little experience had been accumulated in its use.

Dr. Meyerhof could see no advantage in the term "organic terrain" rather than "peat". Mr. Legget pointed out that the term "muskeg" was used a great deal in Canada; "organic terrain" included both muskeg and peat.

Mr. Chapman commented that he considered that when a soil is identified, it is in a sense classified. He could see no objection to the word classification as long as it was recognized that the limits were arbitrary. Mr. Legget agreed that we all classify soil but he objected to the term because a large number of engineers do not recognize the fact that the class limits are arbitrary.

Regarding the length of the document, Mr. Legget suggested that two documents be published. A short one, of one or two pages if possible, would contain all the essentials for field identification and would be widely distributed. The longer document would contain all the necessary explanations as to the reasoning behind what the short document contains. This suggestion was generally approved. In reply to several questions, Mr. Legget stated that the short document would be primarily for field engineers who are not too familiar with soil terminology. It would not be aimed at soil engineers who have had special training, and who probably do not need such a guide.

Dr. McLeod then suggested that the document presented at the conference be published by the Division of Building Research or the Associate Committee. Regarding the short document, he suggested that a committee representing various agencies concerned with soil work be formed. This committee would prepare the short document and have it ready to submit to the next conference for approval.

Dr. McLeod moved the following motion, which was seconded by Dean Hardy.

"Resolved that the following committee be appointed to work with Mr. Legget and Mr. Eden for the purpose of finalizing one or more brochures covering the description, identification, and classification of soils, that will be of maximum usefulness to the various fields of engineering in Canada where soils are used as construction materials;

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

Mr. Brownridge, Ontario Department of Highways, Toronto.

Mr. N.D. Lea, Foundation Engineering Corporation of Canada, Montreal.

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

Mr. L.J. Chapman, Ontario Research Foundation, Toronto.

Dr. N.W. Radforth, McMaster University, Hamilton.

Dr. A. Leahey, Dept. of Agriculture, Ottawa.

Prof. J. Hurtubise, Ecole Polytechnique, Montreal.

This committee will have the power to add to its membership if it appears desirable to do so."

Section 4

General Business

The chairman explained briefly the activities of the Associate Committee on Soil and Snow Mechanics and reported that a meeting of the Soil Mechanics Subcommittee had been held in September. Many of the items to be discussed originated in the Soil Mechanics Subcommittee meeting.

(1) Proposal for Subcommittee on Canadian Landslides

The chairman reported that the proposal for this new subcommittee originated from the Montreal group. The original intention was to study the occurrences of flow type slides which occur in the sensitive marine clays and silts of the St. Lawrence Valley. At the twenty-second meeting of the Associate Committee, it was decided that the study of landslides should eventually include occurrence in all parts of Canada. At first the subcommittee would collect any information available on the slides in eastern Canada.

Membership of the Subcommittee would include:

Mr. N.D. Lea, Foundation of Canada Engineering Corporation

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

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

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

Prof. A. Baracos, Winnipeg, University of Manitoba

Mr. W.A. Trow, Toronto, H.E.P.C. of Ontario

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

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

Mr. J.C. Chagnon, Quebec, Quebec Streams Commission

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

Mr. M. Bozozuk, National Research Council, Ottawa.

(2) Directory of Commercial Drilling Contractors

The Chairman stated that it had been suggested that it would be useful to have a directory which would list all contractors in Canada with equipment and personnel to obtain samples of soil. Many requests for such information had come to his attention. As proposed, the directory would be made up in two sections; (a) contractors with necessary facilities to obtain undistorted samples of the soil stratum, and (b) contractors in category (a) who also could supply qualified engineering or geological supervision.

The chairman asked the conference for comments.

Mr. Peterson endorsed the directory and added that a useful addition would be a directory of laboratories where soil testing could be conducted.

Mr. Peckover stated that in the Directory of Commercial Testing Laboratories, published by the Division of Building Research, many companies stated that they could do soil testing, but they did not have complete facilities for all engineering soil tests, and therefore qualifications must be added to the testing section.

Mr. Pugh considered such a directory would be very useful. Mr. Lea thought that more qualifications should be added to the commercial drilling directory. He stated that it would be possible for any well driller to buy a $50.00 sampler head and thus be listed in the directory.

Dean Macdonald reported that in Winnipeg, no one company had the facilities to obtain samples and do the testing but there were facilities in Winnipeg for carrying out a complete soil investigation. He asked how the directory would list such a case.

The chairman thought that if those laboratories which could conduct the essential engineering tests on soils were included, facilities in any one case would be known. He then asked if the conference considered a directory of commercial sampling contractors and laboratories equipped to do engineering tests on soils would be useful. The general opinion was that it would be.

(3) Maritime Regional Soil Mechanics Conference

The chairman reported that consideration was being given to the holding of a regional conference in the Maritime provinces. This was subject to approval of the Associate Committee. At the Soil Mechanics Subcommittee meeting in September, Professor McFarlane had been asked to determine what interest there would be in such a regional conference.

Professor McFarlane had sent out a circular letter to the Public Works Departments of New Brunswick, Nova Scotia, and Prince Edward Island, to the various federal agencies, to local branches of the Engineering Institute of Canada and to as many contractors and architects as possible. All replies had not been received but replies to date indicated that interest was fairly keen. Amherst or Fredericton were suggested as sites in the circular letter and replies indicated Fredericton would be satisfactory. The University of New Brunswick would celebrate the centenary of engineering in 1954, and thus the University could incorporate the conference as part of those celebrations. April or May were indicated as the most suitable time. Since soil mechanics was a subject not generally known to most engineers in the Maritimes, the first of the two days should be spent on fundamentals. Replies to the circular letter indicated that the second day might well be divided between problems dealing with highway work and deep foundations.

The conference endorsed the Maritime Regional Conference.

(4) Visit to Canada of Dr. Skempton and A.W. Johnson

The chairman reported that Dr. Skempton would be in North America in May or June and had agreed to pay a short visit to Canada. An official invitation had been extended to Dr. Skempton, but it appeared that because of the short time available, his visit would have to be confined to Central Canada.

Mr. A.W. Johnson, Soils Engineer of the Highway Research Board, may be able to visit Canada again this year. His first visit included Montreal, Ottawa, and Toronto; his second, Manitoba, Saskatchewan, and Alberta. The forthcoming visit would be to the eastern part of Canada.

(5) Publications

In the past all publications of the Associate Committee had been sent automatically to everyone on the mailing list. The chairman stated that this had caused a considerable expense because the mailing list had become so large. Since some of the publications were not of interest to everyone on the mailing list, a reply card system, notifying everyone of the publication, would be initiated.

The chairman drew the attention of the conference to Géotechnique, a journal devoted exclusively to Soil Mechanics. Everyone who was not familiar with it was invited to examine the copies on display.

(6) Reports of Regional Representatives

(a) The Maritimes: reported by H.W. McFarlane. There was no organized group in the Maritimes as yet, but interest was increasing. The highway departments of Prince Edward Island and New Brunswick had taken steps to give more emphasis to soil mechanics in the design of roads. Mr. McFarlane thought the Maritime Regional Conference would do much to stimulate interest.

(b) Montreal - reported by N.D. Lea

The Montreal Group had eight meetings since the last conference. The soil mechanics group usually met at luncheons, and were joined in the evening meetings by the Civil Section of the Montreal Branch of the Engineering Institute. Meetings included a discussion on "Earth Flows of St. Thuribe, Quebec," a discussion on underpinning with Mr. Edward White, a discussion on Leda Clays led by Dr. V.K. Prest and H. Gadd, a talk by Dr. Armand Mayer on soil mechanics research in France, a discussion on interpretation of aerial photographs with Professor Donald Belcher, a discussion on bearing capacity of piles led by Dr. G.G. Meyerhof, and a discussion on the Third International Conference led by Messrs. Lalonde and Lea. Mr. C. Brodeur had been elected chairman of the group for 1954.

(c) Ottawa - reported by E.B. Wilkins

Following the Sixth Canadian Conference in Winnipeg, the Ottawa Group held six meetings. They included talks by Mr. Binks on the Trans Canada Highway, Mr. Lea on determination of shear strength of soil, Dr. Northwood on ground vibrations. The highlight of the season was the meeting with Dr. Armand Mayer of France at which over 40 persons were present. In October, Messrs. McRostie and Schriever discussed the Third International Conference and in November the members of the Geological Survey of Canada outlined the pleistocene geology of the Ottawa - St. Lawrence Valleys.

(d) Toronto - reported by M. Davis

The Toronto group had held three evening discussions during the year: Mr. Trow outlined the problem of seepage around dams; Mr. Watson discussed the corrosion of underground services; and Dr. Mayer gave a similar talk to that given in Ottawa and Montreal. Mr. Davis reported that as the Toronto group had trouble in arranging for speakers with fresh topics it was considering starting seminars on various aspects of soil mechanics.

(e) Prairie Provinces - reported by Dean A.E. Macdonald

While there were no organized groups in the Prairie Provinces, there was activity in soil mechanics. In Alberta, the University of Alberta was conducting an active research program. In Saskatchewan, both the University of Saskatchewan and P.F.R.A. were active in the field. A refresher course on concrete and soil mechanics sponsored by the P.F.R.A. was attended by about 45 engineers. In Manitoba, the Soil Mechanics Laboratory of the University of Manitoba was in full operation and was asked to do considerable testing from outside agencies. Professor Baracos delivered two talks on soil mechanics to the Winnipeg and Brandon Branches of the Engineering Institute.

(f) British Columbia - reported by C.F. Ripley
There was little to report for the year from the Vancouver area. During the next few weeks, organizational meetings were planned to establish a soil mechanics group in the Greater Vancouver area in the coming winter. The University of British Columbia was planning to start courses in soil mechanics. Highlight of the year was the address of Dr. Armand Mayer to a joint meeting of the B.C. Association of Professional Engineers and the Vancouver Branch of the Engineering Institute.

SESSION OF DECEMBER 11, 1953

Section 5

Notes on Soil Conditions and Related Engineering Problems in British Columbia

by C.F. Ripley

From the point of engineering use, the variation in soil types and conditions within British Columbia is wide as compared to many other sections of Canada. Topography, climate, and geographical location contribute to this. The steep land gradients, high precipitation, mountain ice caps and ocean boundary contribute to intense activity of the geological processes of weathering, denudation, transportation and deposition of soils in the present and recent past.

The groundwater conditions are of particular importance to the engineer with respect to soil problems. Two major factors which affect the groundwater conditions in B.C. are:

i. The high relief and steep land gradients;

ii. The local climatic conditions.

These factors may contribute to artesian and excess pore water pressure conditions in the subsurface layers which may act either continuously or which may fluctuate in cycles. The pore water pressures have a definite influence on the engineering properties of a soil deposit. Evaluation of the long range stability and supporting capacity of the soil therefore necessitates full consideration of the groundwater conditions.

For purposes of description of the engineering properties and use, the various soil types have been grouped very generally according to their physical characteristics. The occurrence, engineering properties, and problems associated with them are briefly described.

1. Glaciated Deposits

(a) Occurrence

The deposits referred to in this group are arbitrarily restricted to those which have been preconsolidated by the ice pressure during periods of glaciation. They are widely distributed throughout the province, being one of the most predominant groups. The group includes deposits of glacial till, and alluvial and glacio-fluvial soils which have been preconsolidated to a very dense state.

(b) Properties

- Very high density;
- High unconfined compressive strength - generally greater than 60 p.s.i.;
- Low compressibility;
- Fines consist predominantly of silt, very little clay;
- Typical glacial till properties:

Gradation -	gravel sizes	- 30% - 50%
	Sand sizes	- 30% - 50%
	Silt sizes	- 20% - 40%
	Clay sizes	- 0% - 10%

Plasticity of fines - non-plastic to low plasticity;

Unit weight wet - 145 lb. per cu. ft.

Unit weight dry - 130 lb. per cu. ft.

Natural water content - 12%

The typical glacial till is often referred to by contractors as hardpan.

(c) Problems

The glaciated deposits provide excellent foundation support in most instances; natural slopes standing steeper than 1 to 1 for heights in excess of 100 feet are relatively common (a tribute to the strength of the material); resistance of the glacial till deposits to water erosion and weathering is astounding.

The glacial till is well graded from gravel to silt sizes and is thus an excellent construction material for impervious and semipervious embankments; high densities may be obtained with little compactive effort provided the water content at compaction is suitable.

The material is very sensitive to water content, as a construction material, due to the silty nature of the binder; the range of water content within which it may be handled in the field is very low; with a variation of about 5 per cent in water content, the material changes from an excessively dry state to an overly wet state in which it is unstable under traffic of construction equipment. This poses a serious problem for use of the material in compacted fills in the wet coastal climate belt; figuratively speaking, the soil becomes too wet for compaction and unstable under construction traffic if a cloud passes overhead, and conversely, dries very rapidly if exposed to direct sunshine.

2. Gravel and Sand Deposits

(a) Occurrence

Granular deposits in the sand and gravel sizes are a predominant soil group throughout the province; the group includes glacio-fluvial and alluvial deposits widely distributed in river valleys and coastal regions.

(b) Properties

The materials range in gradation from well-graded to uniform depending upon the method of deposition and extent of sorting; they are generally clean and free of fines; the individual particles are predominantly angular.

The deposits provide abundant sources of excellent construction materials for roads, airfields, and embankment construction; the angularity and freedom from fines permit use under abnormally wet construction weather.

Suitability for foundation support varies, dependent upon the density and other properties of the deposit; densities ranging from loose to dense have been encountered; in general, the deposits provide few problems, either as construction materials or for foundation support.

3. Silts, Fine Sands, and Silty Clays

(a) Occurrence

Extensive beds of silts, fine sands, and silty clays are encountered in the interior river valleys and lakes in the arid belt - Kamloops, Kelowna, Penticton districts and central sections of Fraser and Thompson River Valleys; the beds have been deeply incised by the rivers and in the arid belt they stand on high steep slopes above the present river channels; this group has been subdivided into formations of thick uniform beds and of thinly laminated or possibly varved beds.

1. Thick uniform beds.

(a) Properties

Loess-like - reported by R.M. Hardy, Engineering Journal, Vol. 33, No. 9, Sept. 1950.

Low natural water content - 5 to 10%.

Low unit weight - dry - 70 to 80 lb. per cu. ft.

High vertical permeability.

High compressibility.

The material reported by Hardy was predominantly of the silt sizes; it exhibited low compressibility at its natural water content but high compressibility when saturated, even under its own weight.

Somewhat similar properties were obtained for a deposit of very fine sand encountered near lake level at Kamloops. However, this material exhibited unusually high compressibility at its natural water content and when saturated possessed the following properties:

D_{10} - 0.015 to 0.07 mm.

Coefficient of uniformity - 3 to 4

Unit weight - wet - 83 lb. per cu. ft.
dry - 78 lb. per cu. ft.

Natural water content - 6%

Specific gravity - 2.75

Compressive index - dry and saturated condition - 0.4.

(b) Problems

Large landslides in arid belt following periods of unusually high precipitation;

Spectacular subsidences of foundations for buildings and irrigation structures on saturation;

The treatment for the silty material appears to be to prevent penetration of water by protection of natural cover or by application of an impervious covering membrane;

For support of foundations on the sandy material compaction of the sand in place or use of piling appear to be essential to avoid excessive settlements.

ii. Thinly laminated beds.

The deposits are fairly extensive in distribution throughout the arid belt. They have the appearance of varved clays and stand on very steep slopes in the incised river valleys.

(a) Properties

Stiff, low compressibility when dry, stable when not subjected to excess hydrostatic pressures.

(b) Problems

Stability of slopes and very large landslides associated with periods of unusually high precipitation and with irrigation of elevated river terraces.

4. <u>Sensitive Clays</u>

(a) Occurrence

Marine deposits in fjords and river mouths near and below sea level on the mainland, and on the southern tip of Vancouver Island near sea level.

(b) Properties

Normally consolidated except where exposed to desiccation.

Extra sensitive, sensitivity index range 6 to 12.

Natural water content 10% to 20% above liquid limit.

Liquid limits range to 100.

Liquidity index ranges to 2.7.

Unconfined compressive strength 5 to 9 p.s.i.

Consolidation test data --

Compressive index range 0.3 to 2.6

Breaking point stress exceeds existing overburden stress by about 0.3 ton per sq. ft.

(c) Problems

Very difficult foundation material; sharp reduction in strength and high compressibility when "breaking point" stress is exceeded.

5. Highly Plastic Clay Shales

(a) Occurrence

Generally sedimentary rock formations, often compaction shales of marine deposition.

Fort St. John - Fort Nelson Area - Lower Cretaceous Fort St. John Group.

Cariboo District - Miocene, Fraser River Formation.

Greater Vancouver area - Kitsilano Group.

Princeton Area.

(b) Properties

Loss of strength and breakdown of soil structure due to one or all of, shrinkage on drying, swelling on exposure to water, freezing, and reduction of overburden pressure.

High volume change with changes in water content with accompanied high swelling pressures.

(c) Problems

Stability of slopes - very large slump blocks in these materials can be seen in the Vancouver, Princeton, Quesnel and Fort St. John-Fort Nelson area; hummocky terrain indicating slow creep of the upper weathered mantle on slopes of this material is common.

Bridge abutment problems.

6. Highly Organic Soils

(a) Occurrence

Coastal regions, river deltas, intermountain plateaus, extreme northern areas along northwest highway system; peat bogs, forest floor debris and highly organic silts.

(b) Properties

High compressibility.

(c) Problems

The wide distribution of these soils creates difficult foundation problems for support of buildings, highways, railways, etc. in the developed regions of the province.

7. Miscellaneous

Brief mention is made of two other types of soil deposits which are frequently encountered throughout the province and which present their own peculiar problems to the engineer - these are slide debris and talus.

The deposits of slide debris are relatively common in valleys throughout the province; the gradation and structure vary with each deposit; these deposits are commonly traversed by roads, railways, transmission lines and pipe lines; the general problem associated with them is the stability of slopes.

Deposits of talus adjacent to the wall of mountain slopes are very common; they often provide a means of access for drainage from rock slopes above and into the subsoil. In this way they contribute to the occurrence of continuous or fluctuating pore water pressures in previous soil layers below the ground surface.

Discussion

The chairman remarked that Mr. Ripley had given an excellent cross-section of the various soil conditions in British Columbia, and that his paper showed that no one province had a monopoly on soil problems.

As an example, Mr. Baracos described swelling clays at Vernon in the upper Okanagan Valley where there was a low rainfall. A housing development on a side hill had experienced difficulties caused by swelling soils. Some of the houses were constructed without basements. As no drainage had been provided for the crawl space under the houses, water collected in the crawl space and caused the clay to swell. In some cases the houses were severely damaged by the swelling. In the same area, swelling of the clay had caused many breaks in asbestos cement water mains. Sand backfill around the water mains had to be used to overcome the swelling problem.

In reply to a question from Mr. Peterson, Mr. Ripley stated that the shales in the Vancouver area were designated by the names Burrard and Kitsilano, but he was not sure of their period. In the Cariboo district the shales were of Miocene age and were designated as the Fraser River Formation. In the Fort St. John Area, the shales were of Lower Cretaceous age and were similar in many respects to the Pierre and Bearpaw shales of Saskatchewan. Dean Hardy remarked that the geologists would not agree that the Fort St. John shales were the same as the Bearpaw shales. Mr. Ripley stated that the engineering properties were very similar.

Dr. Radforth asked whether the organic materials which had been submerged were of sedimentary or detrital origin. Mr. Ripley said that there were sediments in the Kitimat area; the tidal flats supported a thick growth of marsh grass. Silt is deposited from the rivers. The coast line has been subjected to a number of geological depressions and elevations, which would account for the buried beds of organic material.

Mr. Torchinsky asked whether the sensitive clays had been desiccated and what their preconsolidation load would be. Mr. Ripley replied that in the Victoria area, the clays had been generally desiccated throughout. The preconsolidation pressure had not been determined accurately, but pressures of 16 ton/sq. ft., the capacity of the testing apparatus, had been attained without determining the preconsolidation load. Mr. Ripley then described an example of a clay-filled ravine in Victoria. The clay was about 100 feet in depth and only the top 10 feet had been desiccated, but other ravines were completely desiccated. The variation in conditions was caused partly by the ground and rock surfaces, and partly by the uplift and submergence of the coast line.

Section 6

Research Underway in Department of Civil Engineering, University of Alberta

by R.M. Hardy, Dean of Engineering

Reference has been made at previous meetings to the program underway at the University of Alberta for the past six years concerned with the treatment of frost-susceptible soils with "lignosol" to prevent frost heaving. One treated location has now been observed over three winters. Some deterioration in the effectiveness of the treatment was noted during the third winter. This is due to the fact that the lignosol is water soluble and therefore is gradually leached out of the treated zone.

Recent work at Cornell University under a U.S. Navy Research contract has shown that it is possible to permanently stabilize lignosol by the addition of chromate salts. An insoluble gel is produced. At present, work is being carried out on the application of this finding to the problem of frost heave. On a laboratory scale very satisfactory results have been secured. The lignosol and chromate admixture can be mixed with the soil in solution. The mixture then gradually sets up into a gel. By varying the percentage of the chromate admixture the setting time of the gel can be controlled.

Preliminary laboratory tests show the lignosol-chromate gel is equally as effective in preventing frost heaving as is the plain lignosol, although it is likely that the mechanism is quite different. In addition, it has a marked stabilizing effect on the soil. The addition of the chromate increases the cost somewhat but this will be more than offset by the advantages of a permanent treatment. Effective methods of injecting the chemicals into the soil still present a major problem. On a laboratory scale it has been found possible to move chemicals such as lignosol, calgon, and aerosol through fine-grained soils by the process of electro-osmosis.

An interesting effect has also been observed with flow under combined hydraulic and electro-osmotic heads. The velocity distribution for liquid flowing through a pipe under a hydrostatic pressure is parabolic in form, being zero at the pipe wall and a maximum at the centre of the pipe. The velocity distribution across a small opening in a liquid subjected to an electro-osmotic head is generally assumed to be a maximum at the "double layer" at the surface between the liquid and solid phases, and to vary, perhaps parabolically, across the opening with the minimum velocity being at the centre of the opening.

The variation in velocity across the openings in both cases is considered to be due to the viscosity of the liquid. On analytical grounds it appears possible that a combination of hydrostatic and electro-osmotic heads would tend to neutralize the viscosity effect and give a resulting velocity that would be greater than the sum of the velocities of flow with each pressure acting separately.

Laboratory tests have confirmed this reasoning. Up to twice the velocity of flow has been measured in silty type soils under combined hydrostatic and electro-osmotic heads as compared to the sum of the velocities occurring with the pressures acting separately.

In the soil types most highly susceptible to frost action the flow of liquid is still very slow under either hydraulic or electro-osmotic gradients of practical magnitudes. However, the combination of the two presents some possibilities of practical application in the frost treatment problem. These are still being investigated and it is planned to try a field installation in the near future.

Interesting results have also been secured on a laboratory scale in stablizing a highly plastic clay soil by electro-osmotic treatment using iron electrodes. The soil treated had liquid limits ranging from 45 to 107 and plasticity indices from 27 to 78. By electro-osmotic treatment the moisture content vs. log of strength curve was shifted considerably towards the greater strength area of the plot. The shift in the curve represented an increase in strength of as much as 300 per cent. Soaking of the samples indicated that the greater portion of the strength was retained.

A field installation has shown the same trend but to a considerably lesser degree.

To appreciate the potentialities of the process of electro-osmosis it is necessary to understand the difference between the use of the process in stabilization as distinct from control of seepage forces. Dr. Leo Casagrande, now of Harvard University, is the world authority on the application of the principles of electro-osmosis to engineering problems. He has worked both in the fields of seepage control and stabilization. However his successful practical applications have almost all been a matter of seepage control. This was the principle used in the recent installation at Flint, Michigan, which is presently being so widely reported in the trade journals.

We recently successfully used the principle of electro-osmosis in seepage control at Unity, Saskatchewan, on a mine shaft 7 by 12 feet in cross-section. The soil profile was 16 feet of clay overlying a bed of water bearing fine sand which in turn was underlain by clay at a depth of 22 feet below the surface at one corner of the shaft and $23\frac{1}{2}$ feet at the opposite corner. The original plan was to cement grout the sand layer. However, after several weeks of effort the grouting procedure was abandoned because the sand was too fine to take the grout. Since the dimensions of the shaft and sequence of surface works had been decided upon, assuming the grouting process would be successful, other conventional methods of handling the problem were precluded except at considerable expense.

An electro-osmosis installation to control the seepage forces and eliminate the seepage flow and quicksand action offered a considerable saving in time and expense as compared to any other procedure that seemed feasible. Canvas-wrapped wire-mesh well points were placed on a spacing of 6 feet around the shaft, placed at an angle of about 45 degrees and extending from the top of the sand layer to the underlying clay. These acted as cathodes and pipe anodes were driven midway between the cathodes.

The installation was powered with two portable welding units connected in series. This installation successfully controlled the seepage for a depth of 6 feet. This permitted the shaft to be carried to the underlying clay at the shallow corner. The installation was not successful for the remaining $1\frac{1}{2}$ feet at the deepest corner.

In conclusion I wish to comment on the remarks of the session reporting on the Third International Conference in Switzerland in which application of the process of electro-osmosis was described as an art because the fundamental nature of the process is not understood. Admittedly it is highly desirable to acquire a better understanding of the fundamental nature of electro-osmosis. However it is submitted that the same statement is equally applicable to the whole question of the shearing strength of cohesive soils. Yet numerical shearing strengths of soils are essential to the analytic approach to the solution of many practical soil problems in engineering practice. Similarly the known principles of electro-osmosis have significant potentialities in engineering practice. While there is room for the accumulation of much data that will permit its rational application to practical problems, it is submitted that this can proceed without waiting for a complete understanding of the fundamental nature of the process.

Discussion

Dr. Meyerhof remarked that he had the good fortune to be on the staff of the Building Research Station in Great Britain when Dr. Leo Casagrande worked there for four years. Dr. Meyerhof thought that Dr. Casagrande would agree with Dean Hardy that the fundamental aspects of electro-osmosis are not yet fully understood. More research is needed in this field by physical chemists. In the meantime empirical information from engineering applications should be collected. Dr. Casagrande had been successful with applications in Germany, Norway, Great Britain, and now, in the United States. The main application of electro-osmosis had been in controlling seepage forces in silts.

Dr. Meyerhof reported that electro-osmosis had been tried on three practical cases in Great Britain, namely: in the excavation of a silty soil where trouble had arisen, in the construction of a culvert*), and in the treatment of an unstable clay slope. The latter two cases illustrated that the effect of electro-osmosis is localized around the electrodes. For this reason, he was not optimistic about Dean Hardy's attempt to move chemicals a great distance into clayey soil by electro-osmosis. Dr. Meyerhof was of the opinion that the use of laboratory experiments was limited in practice because the localized effect would not be evident in the laboratory. Another factor which should be considered was that of costs. Electro-osmosis was an expensive process.

Dean Hardy agreed with Dr. Meyerhof that the greatest, and possibly the only success, of electro-osmosis had been in controlling seepage forces in silts. Regarding the shaft mentioned in the paper, electro-osmosis was the cheapest method under the circumstances. If the soil conditions had been known beforehand, other methods such as sheet piling or well points could have been used. But with the head frame of the shaft already in place, the other alternatives were ruled out.

Dean Hardy could not agree with Dr. Meyerhof that the effect of electro-osmosis was confined to a local area around the electrodes. In the field trials, with electrodes spaced at 15-foot centres, samples were taken at the anode, midway between the electrodes, and at the cathode. In this soil, there was a measurable increase in strength in the samples taken midway between the electrodes, indicating that in tests conducted by Dean Hardy the effect was not confined to the soil immediately adjacent to the anode. In silts there is no stabilization effect, but in plastic soils those

*) Graham, J. "The Reconstruction of Culvert No. 146 near Ayton." Inst. Civil Engrs., Railway Engg., Div. Paper No. 42, Jan. 1951.

used in the studies of the injection of chemicals, results indicate an increase in strength. Regarding the use of electro-osmosis in the injection of lignosol, it is used in conjunction with hydraulic pressure to disperse the chemical in the soil.

Mr. J.L. McFee asked why wire-mesh cathodes were used instead of well points. Dean Hardy replied that the radius of influence of a well point would be too small in this case as the soil was a fine sand, but its permeability was relatively low. Dean Hardy had never seen a material which behaved quite like the one in question.

Mr. White stated that at a job in Michigan, the sheet piling around the excavation had been moving. When electro-osmosis was applied, the movement of the piling was halted. Mr. White considered this a stabilisation of the silt. Dean Hardy replied that the movement was caused by seepage forces and when this force was controlled the movement stopped. The silt itself need not have been stabilized. With clays, laboratory experiments indicate a change in the properties of the soil.

Section 7

Stability of Dams on Highly Plastic Clays

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

The leaders in the soil mechanics field have always emphasized the complex nature of clays and the fact that many problems are not subject to mathematical analysis because it is impossible to completely duplicate field conditions in laboratory tests (1). Of the soils encountered in Western Canada, the highly plastic clays often appear to be uniform and as laboratory test results can be duplicated it is commonly assumed that stability problems in these materials can be analysed using laboratory strength tests. However, experience over the past twelve years indicates that conventional methods definitely give results on the unsafe side when applied to earth dams on saturated highly plastic clay foundations. Numerous cases have been encountered where stability studies for dams located on soft to medium highly plastic clays have indicated safety factors in excess of 2 or 3 and where slides through both fill and foundation have occurred subsequent to construction. In these cases the shear strength had been based on quick tests or half the unconfined compressive strength. The plasticity characteristics of the foundation soils ranged from a liquid limit of 60 at a plasticity index of 40 to a liquid limit of 110 at a plasticity index of 80. The water content range was 30 to 60 per cent with the corresponding shear strength 15 psi to 3 psi. The soils can be described as soft to medium with no visible evidence of stratification.

In most cases fairly complete soil studies were made previous to construction. However, in view of the importance that these experiences are assuming in present and future studies, all movements are being reinvestigated and a thorough analysis will be made of each. In addition to the recovery of chunk samples and 3-inch and 5-inch Shelby tube samples for testing, a vane borer and a field unconfined compression unit are being used to define more completely the strength characteristics.

It would appear that the reasons for the discrepancies between predictions based on laboratory tests and actual field behaviour are due to one or more of the following:

1. Reduced strength under sustained loading or at slow rates of strain

Casagrande and Wilson (2) have performed what they describe as "creep tests" and they conclude that if certain saturated clays are subjected to sustained loading at constant water content, the strength is only a portion of that given by ordinary unconfined compression tests or quick triaxial tests. In a creep test the sample is set up in the same manner as in an undrained triaxial test with special precautions to insure a constant water content. An axial stress is then left on the sample until failure occurs. In the P.F.R.A. laboratory a considerable number of samples have been tested in the manner indicated and it has been found that for sustained loading up to 60 days the strength may be only one-half that obtained by quick tests. Normally when a clay layer is loaded and consolidates there is a reduction in water content and a gain in strength with time. However, in the case of very thick layers the consolidation and strength gain near the centre of the layer may be negligible for a long period. In fact the creep tests indicate that there is a reduction in strength if the water content remains constant. It would therefore appear that the reduction in strength at constant water content is of serious practical significance for thick clay layers. In a paper by Tan Tjong-Kie (3), various rates of strain were studied and it was found that for highly plastic clays a decrease of ninetyfold in the rate of loading resulted in a decrease of about 50 per cent in the strength. Terzaghi and Peck(4) state that if the shear stress exceeds approximately one-half the maximum shear strength, creep may go on indefinitely. Tschebotarioff(5) notes similar behaviour.

From the above it would appear that creep or movement may occur if this type of soil is stressed beyond 50 per cent of the maximum shear strength. Field behaviour appears to bear out the results of laboratory creep tests as cases are known where movement occurred four years after construction.

2. Reduced strength due to pore pressures acting on fissures, stratification planes or partings within the clay

The pore pressures might be induced by the weight of the dam or as a result of reservoir pressure. Often the minor geological(6) details such as fissures or stratification are difficult to detect even in large diameter borings or test pits. It is therefore impossible to evaluate the shear strength and stability under such conditions by laboratory tests. About the only practical way is to base stability computations on pore pressure observations while the structure is being built. Terzaghi(7) has stated that under adverse conditions the strength as determined in the laboratory may be as high as five times the actual strength realized in the field because of the fact that it is impossible to duplicate conditions in a laboratory test.

3. Progressive failure

The stress-strain characteristics of the foundation material may not be compatible with the embankment material and progressive failure may result. Where the foundation is soft and plastic large strains may occur in the foundation at low stress and result in rupture of the more brittle fill material. In other words, the strength of the two materials will not be mobilized together and hence the true factor of safety will be lower than the computed factor of safety based on the assumption of complete mobilization of the shear strength on the failure arc. A case is known where a dyke of constant height on approximately the same type of foundation varied in composition from a lean clay placed fairly dry to a medium plastic clay at a considerably higher water content. It appears that the failures through the fill and foundation have been confined mostly to the area where the dyke was composed of the drier and more brittle clays. That is, where the dyke and foundation both consisted of the soft weak material the overall structure has been more stable than where the dam was constructed of dry, brittle material and the foundation was soft plastic material. This would appear to substantiate the more progressive failure theory. Tschebotarioff(5) suggests a method of choosing strength values which allows for the different stress-strain characteristics of the dam and foundation. It is also possible that foundation spreading may have produced a tension failure in some of the fills.

These experiences are rather disturbing and would seem to call for a modification of the approach to stability problems in highly plastic clays. It would appear that stability evaluation based on ordinary laboratory strength tests can only be used as a rough guide and when this is done, very conservative factors of safety should be used. It may very well be that with further research on creep tests and stress-strain characteristics a more accurate appraisal will be possible. However, until such time as this is done it is suggested that laboratory tests be supplemented by careful field observations, particularly on the important embankments. In the case of minor structures where slides are not serious the most economical procedure no doubt will be to base the design on laboratory strengths and general observations of field behaviour. For the more important dams a complete system of piezometers (8,9) should be utilized to measure pore pressures within the foundation. In addition, apparatus should be installed to measure movements in both the fill and foundation. These should include vertical movements (9), lateral movements at right angles to the centreline, and longitudinal movements parallel to the centreline (10). It is thought that a study of these measurements can warn of the development of any critical conditions so that the design can be modified or the rate of construction altered to suit the unpredictable pore pressures. In addition relief wells or sand drains are often useful in dissipating high pore pressures.

In searching the literature for information regarding stability of highly plastic clays it appeared to the writer that in some respects the soil mechanics literature may not place the limitations of existing methods in their proper perspective. While most texts mention briefly the limitations or shortcomings those readers who are inexperienced may fail to recognize these warnings and to rely upon theory, laboratory tests, and a mathematical approach. In the case of technical papers it would appear that many authors are inclined to go into print only when they have been successful in checking some theory or method. When they have had an experience that seemed to contradict existing theory and methods they have perhaps tended to avoid publicity. In addition, of course, there is always the natural tendency to conceal difficulties and failures. One is reminded of a statement made by Dr. A. Casagrande in his closing remarks of a recent paper (11): "A scientific approach to this subject developed from infancy to a young adult, proud of its importance, but perhaps too often unaware of its limitations".

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(9) U.S. Bureau of Reclamation, Instruction in Installation and Operation of Test Apparatus for Earth Dams, Branch of Design and Construction, Sept. 1950.

(10) Sherard, J.L., Influence of Soil Properties and Construction Methods on the Performance of Homogeneous Earth Dams, U.S.B. of R. Tech. Memo. 643, January 1954.

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Notes on other P.F.R.A. Activities

1. Travers Dam - This key structure on the Bow River Irrigation Project in Southern Alberta, is now being completed. It is a zoned earth embankment 140 ft. in height and containing 4,500,000 cu. yd. of fill material which makes it the largest earth dam in Canada. Extensive test apparatus including piezometers and settlement apparatus is being installed. Of special interest is the chute spillway. Both ends of this structure rest on a swelling type shale and the central portion has been placed on approximately 30 ft. of compacted sand and gravel. Careful observations are being taken to detect any movement.

2. Study of highly plastic materials - This includes the study of highly plastic clays and also studies with respect to clay shales that are of very frequent occurrence in the West. Laboratory tests involving shrinkage and swelling at predetermined stresses are being carried out and an attempt is made to correlate these results with measurements of movement which are being taken on spillways and structures located in the shale. This involves precise levelling on points in the concrete, foundation movement gauges and deep bench marks.

3. Sand filters in dams - During the past year experimental work has been carried out in the matter of installing thin pervious filters in the downstream portion of earth dams to conserve sand or sand and gravel where it is expensive. A horizontal filter layer is placed on the foundation or embankment and impervious material is placed over it to the full width of the section. When the depth has reached 10 or 12 ft this material is trenched to the underlying pervious blanket and the trench backfilled with sand. The process is then repeated.

4. Deep sand drains - A limited number of deep sand drains 8 to 12 in. in diameter and to a depth of 100 ft. have been installed in soft unstable foundations using conventional rotary drills. The purpose is to relieve pore pressure in sandy layers of the foundation. Cost $1.50 - $2.00 per foot.

5. Sounding methods - Experimental work is being carried out using a vane borer and a cone penetrometer to evaluate strength of the softer foundation clays.

6. Seepage control in canals and dugouts - Further experimental field installations have been carried out to investigate materials suitable for lining. The following materials are being studied: thick compacted linings, thin compacted linings with cover, loose earth linings, bentonite linings, covered asphalt membrane, shotcrete, reinforced concrete and concrete blocks.

7. Unstable Foundations - Because of the soft and difficult foundation conditions at several dams now under construction, it has been found necessary to resort to stage construction spreading the embankment construction over 2 to 3 years.

8. Humid Room - A humid room including one area for high humidity storage and temperature control and another area for sample preparations has been construction and is now under test in our main laboratory at the University of Saskatchewan.

9. Permeability Studies - Field tests have always been favoured for permeability determinations. Pumping tests have proved very satisfactory and limited use has been made of drill hole tests and tests in auger holes.

Discussion

Mr. Torchinsky asked what effect the rates of shear used in the laboratory had, at present, on the bearing capacity used in foundation design. Mr. Peterson replied that it was usual to use a safety factor of 2 or 3 in the design of footings, therefore the problem of rate of loading was not as acute as with case of the design of earth dams, where the safety factor was generally 1.5. Also, progressive failure, as described in the paper, does not seem to be so important in footings, as with earth dams.

Mr. Torchinsky then asked if the safety factor should not be increased from 2 or 3, since it is possible that many foundations are being designed with a safety factor of 1. Mr. Peterson stated that if the safety factor has been used up to date, there is no reason why it should not be continued. The theory of bearing capacity has been more or less justified by case records, for instance the Transcona elevator failure as described by Peck*. Mr. Baracos reported that the investigation of the Transcona failure had been continued. Recent investigations show that rate of loading was an important factor in this failure. This would correspond to a rapid loading test in the laboratory. Strength tests were run at a constant rate of stress at a rapid rate. Using the values obtained in this way, the theoretical bearing capacity was found to correspond exactly with the actual bearing capacity.

In the Winnipeg area, there had been a number of landslips along the river banks. Several had been investigated, and the results bear out Mr. Peterson's remarks. From the analysis based on laboratory results, the stability of the banks were 4 or 5 times what failure conditions indicated.

Dean Macdonald noted that the soil was badly fissured along the river banks. In the spring run-off, water enters the fissures, leading to lubrication. He was of the opinion that factors such as fissuring may have an influence on the bank's stability. Dean Hardy reported that fissures had been the cause of some slides in the clay in the Edmonton area.

Dr. Meyerhof asked if in the stability analysis, any account had been taken of tension cracks, and if any fissures or stratifications were in evidence in the clays. He thought that if there were fissures or stratifications, there may have been a progressive local softening, causing an overall reduction in strength of the clays. Mr. Peterson replied that no account was taken of tension cracks in the failures he had discussed. He thought tension cracks were associated more with failures of natural banks. On the point regarding stratifications, no evidence had been found indicating stratifications, but this was not conclusive.

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*Peck, R.B. & Bryant, F.G. - "The Bearing - Capacity Failure of the Transcona Elevator" Géotechnique Vol. 3, No. 5, March 1953.

Fissures had been detected in the clays, but they were always closed. Mr. Peterson was not sure on the point about softening.

Mr. Trow asked if the author would elaborate on the method of determining permeability from boreholes, mentioned in the brochure. Mr. Peterson stated that the method followed was the same as that of the U.S. Bureau of Reclamation.

The Chairman, referring to Dr. Meyerhof's comments, mentioned that he was of the opinion that failures in varved clays at Steep Rock may have been due to softening. Also, the Western Association of State Highway Officials road test in Idaho had several failures which were closely examined. To date no change in the characteristics of the subgrade material had been found which would account for the reduction in strength. Some sections of this road had been subjected to 8500 passes of heavy vehicles without failure, while other sections had failed with less than 100 passes.

Dr. McLeod reported that the failures were confined to the outer 6 feet of the pavement. Although there was no apparent change in soil properties, because the failures were confined to the outer 6 feet of pavement indicated that the surcharge effect caused by the pavement may be a factor. To evaluate this surcharge effect, pavement would be placed on the shoulders of the road to see if that would prevent failure.

Dean Macdonald stated that this type of failure had occurred in highways in Manitoba, and asked Dr. McLeod if the failure may not have been due to a change in precipitation. Dr. McLeod replied that precipitation could not have been the cause because the test site was a desert area requiring irrigation. In reply to a question from Mr. Ripley, Dr. McLeod stated that the subgrade material was a silt. Great care had been taken in the preparation of the subgrade and compaction had been controlled to a depth of 5 feet.

Section 8

FREEZING INDEX DATA INFLUENCING FROST ACTION

by
E.B. Wilkins and W.C. Dujay

It has frequently been reported that air temperature is the most important of the many factors affecting soil temperatures. When considering frost action, a reliable indication of the effects of climate is the "freezing index" which may be defined as a measure of the combined duration and magnitude of below freezing air temperatures based upon a long period of record. The unit employed to express the index is the "degree-day", which represents one degree of declination from a given point (here 32°F) in the mean outdoor temperature for one day. Thus a mean daily temperature of 10°F. would be 22 degree days of frost. The degree day is plus when daily mean temperature is below 32°F. and minus when above. A cumulative total of degree days is plotted against time, and the freezing index determined for the algebraic difference between the maximum and minimum points on the curve. The Corps of Engineers U.S. Army (2) describe a "normal freezing index" which is computed for normal air temperatures based upon a long period of record, usually 10 or more years.

Primarily, this project was undertaken to provide a check on airport runway design. The years 1945 to 1950 were initially chosen to coincide with the runway evaluation program. Weather Observation Records of the Meteorology Division of the Department of Transport were employed as the source of data, and freezing indices for some 120 stations distributed across Canada were computed. A mean figure for the five winters 1945 to 1950 was obtained and plotted on the map. Using these points, contour lines spaced at 500 freezing index were drawn. In parts of the Northwest Territories and northern Quebec, the contours have been "dashed" to indicate approximate location only, as a result of the wide spacing of observation stations. Work is continuing to extend the time period to ten winters for the stations already evaluated. Except for major stations, the only records of the maximum and minimum temperature of each day were available. Calculations at a few stations showed a difference of less than ten per cent between freezing indices obtained from the mean daily temperature and the average of the maximum and minimum temperature for each day.

Legget & Crawford (1) have compiled a freezing index frequency curve for the years 1885 to 1950, a period of 65 years for Ottawa. The mean of the 5-year period 1945 to 1950 for Ottawa, according to our data indicated that 66 per cent of the winters had greater freezing indices during the 5-year period. The Corps of Engineers (2) show a curve of combined thickness of pavement and base required to prevent freezing of the subgrade plotted against the freezing index. Depths of frost measured in sewer excavations in Ottawa have been indicated on this design curve by Legget & Crawford (1).

In conclusion, it is hoped this paper will contribute towards a further understanding of frost action in Canada.

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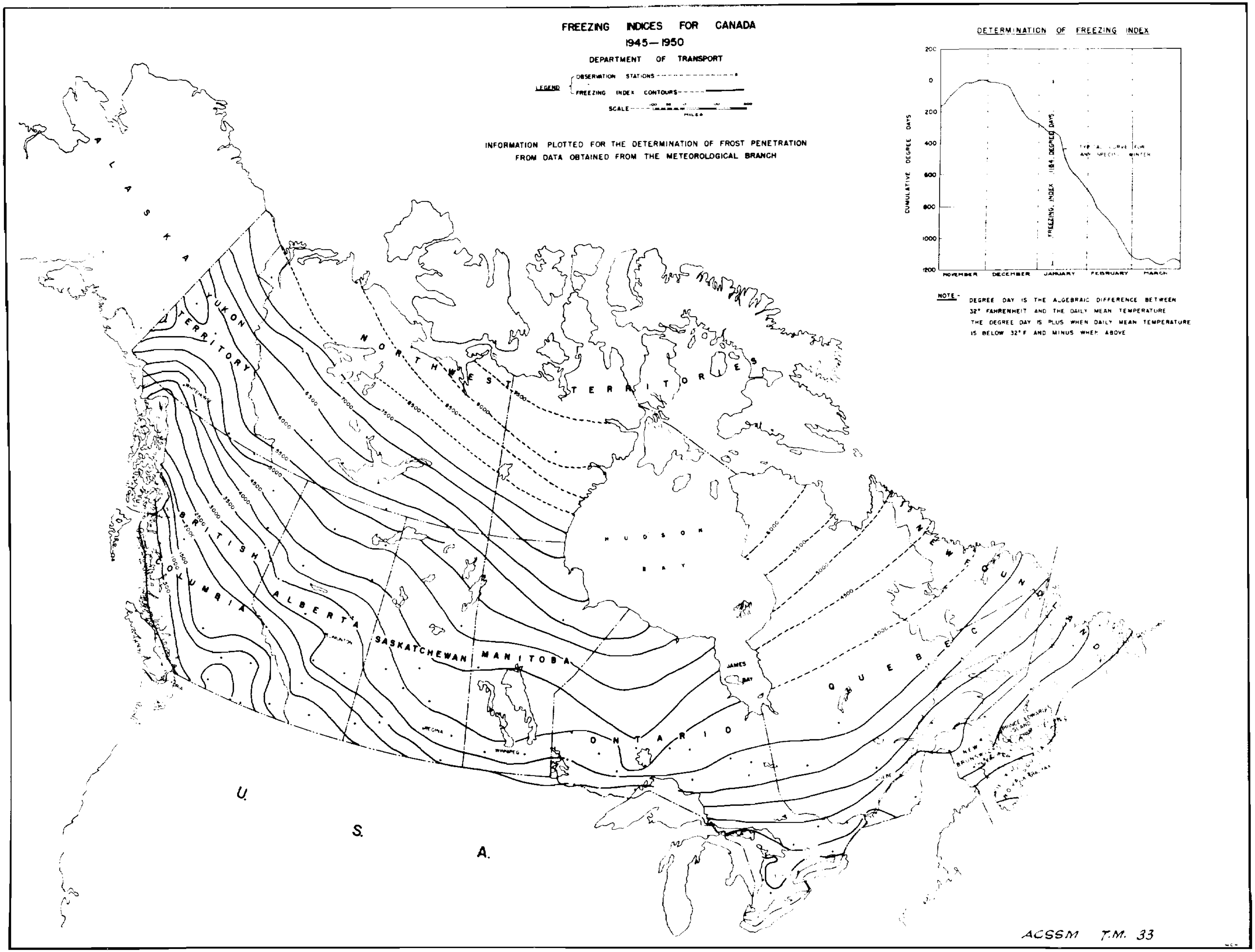
FREEZING INDICES FOR CANADA
1945—1950
DEPARTMENT OF TRANSPORT
LEGEND
OBSERVATION STATIONS
FREEZING INDEX CONTOURS
SCALE
MILES
INFORMATION PLOTTED FOR THE DETERMINATION OF FROST PENETRATION
FROM DATA OBTAINED FROM THE METEOROLOGICAL BRANCH
DETERMINATION OF FREEZING INDEX
CUMULATIVE DEGREE DAYS
200
0
200
400
600
800
1000
1200
NOVEMBER
DECEMBER
JANUARY
FEBRUARY
MARCH
FREEZING INDEX 1184 DEGREE DAYS
NOTE -
DEGREE DAY IS THE ALGEBRAIC DIFFERENCE BETWEEN
32° FAHRENHEIT AND THE DAILY MEAN TEMPERATURE
THE DEGREE DAY IS PLUS WHEN DAILY MEAN TEMPERATURE
IS BELOW 32°F AND MINUS WHEN ABOVE
ALASKA
YUKON TERRITORY
NORTHWEST TERRITORIES
BRITISH COLUMBIA
ALBERTA
SASKATCHEWAN
MANITOBA
ONTARIO
QUEBEC
NEWFOUNDLAND
NEW BRUNSWICK
PRINCE EDWARD ISLAND
NOVA SCOTIA
HUDSON BAY
JAMES BAY
REGINA
WINNIPEG
U.S.A.
1000
1500
2000
2500
3000
3500
4000
4500
5000
5500
6000
6500
7000
7500
8000
8500
9000
9500
ACSSM T.M. 33

Discussion

Mr. Peckover asked at what time in the fall the freezing index computations were started, and if there was a standard procedure in choosing the time for the start. Mr. Wilkins replied that the computations were started about the end of October. Above-freezing weather at the beginning of the winter does not affect the freezing index value since it is an algebraic difference between the maximum and minimum points on the curve. The procedure in the compilation is the same as that used by the Corps of Engineers, U.S. Army.

In reply to a question from Mr. McLaughlin, Mr. Wilkins stated that short periods of thaw were taken into account in the computation of the freezing index. Dr. McLeod agreed that thaw periods should be considered as in warm days there would be less tendency for heat to escape from the ground, which would be reflected in the rate with which the frost line penetrates the ground.

Mr. Baracos stated that soil temperature records indicate a time lag, and thus the frost line could continue to penetrate the ground even though the air temperatures were above freezing. Dr. McLeod agreed with Mr. Baracos that there was a large time lag, but maintained that thaw periods, in spite of the time lag, will be reflected in the position of the frost line.

Mr. Davis reported that soil temperature measurements indicate that the time lag increases greatly with depth. At a 6-inch depth daily variations in air temperature are reflected.

Mr. Crawford agreed with Dr. McLeod that the thawing periods should be considered in the freezing index. Referring to the inset in the map, if thaw periods were neglected the freezing index value would be too large and the curve would be shifted to the left. The time lag is shown by the fact that the frost line continues to penetrate in May or June.

Mr. Chapman asked if the Freezing Index map had been compared with the map of January Temperatures published by the Meteorological Service. The author replied that this had not been done to date, but it might provide an interesting comparison.

Mr. Henderson asked if the southern boundary of permafrost parallels a certain freezing index line. Mr. Wilkins stated that he had not examined the map from that point of view. Mr. Brown stated that he intended to look into the matter in the permafrost survey being undertaken by the Division of Building Research.

Section 9

FIELD INVESTIGATION OF THE EARTH FLOW AT RIMOUSKI, QUEBEC

by

Dr. G.G. Meyerhof

INTRODUCTION

One of the main outstanding problems of the stability of hillsides and slopes is large earth flows of sensitive clays and silts. Some fifty earth flows of this type have been recorded, about half of which occurred in Norway (1) and the remainder about equally divided between Sweden (2) and Eastern Canada (3).

A common feature of these earth flows is their retrogressive nature developing after local disturbance at the toe. Such disturbances may be caused by erosion from a river causing instability and sliding of the soil which, in turn, propagates slipping movements retrogressively towards the top of the slope. The soil movements disturb the sensitive clays or silts to such an extent that they liquefy and flow down the hill like a viscous liquid on a very small slope.

Site and Geology

An earth flow of this type occurred about two years ago near Rimouski, Quebec, on the Rimouski River, a tributary of the St. Lawrence River. The St. Lawrence River Valley is well known for the number of earth flows which have occurred there. The present slide occurred on the west bank at a sharp bend of the Rimouski River, where the wooded bank drops fairly steeply from the level plateau at a slope of about one in four. Immediately to the southeast of the site, an earth flow took place about 100 years ago, while immediately to the northwest, some shaley rock lies close to the surface.

The erosion of the River bank had been noticed for several months previously by eye-witnesses, one of whom had seen a fairly deep undercutting of the bank. Some of this eroded material had been deposited in the form of sand bars in the River at the site; swampy ground existed behind the sand bar and a whirlpool is said to have existed in front of it.

History of the Slide

The earth flow occurred in two stages: a first slide which affected about one-third of the present disturbed area, and a second larger slide, as shown in Fig. 1. The first slide occurred on August 3, 1951, in the evening. It lasted a few, possibly five minutes, taking down the steeper wooded area of the site. Some of the mud blocks were about 15 feet wide and slid into the River blocking it for a distance of about 700 to 800 feet. This slide is said to have been about 400 feet long and 200 feet wide; the width is about the same as that of the swampy ground at the toe. The depth of the slide was about 50 feet near the top, where some springs were discharging.

The second slide took place on August 6, 1951, in the morning, and lasted for about half an hour. The main soil mass descended during the first quarter of an hour and then a smaller quantity of material followed, pieces of soil dropping off from the upper edge during the following few days. The material blocked the River for about half a mile, overflowed both banks and formed a lake of about three-quarters of a mile in length upstream, flooding about a dozen houses and interrupting the road and services. The extent of the affected area is shown in Fig. 1. The debris consisted mainly of silt blocks covered with numerous trees; the blocks had a height up to about 25 feet and rested in a very soft mud which flowed into the River with the blocks. The length of this second slide was about 1,700 feet and its maximum width about 800 feet. The average slope of the affected area, which is about 25 acres in surface, is now about one in eight. About 1 million cubic yards of soil are estimated to have been involved in the earth flow.

Site and Soil Conditions (Summer 1953)

In view of the rough nature of the ground, only an approximate longitudinal section of the earth flow can be given, as indicated in Fig. 2. The upper part shows the slipped soil mass typical of a retrogressive rotational slide. The centre and lower part of the area consists of numerous hard silt blocks resting in a soft matrix of silty clay with a harder upper crust. Since the earth flow occurred, some of the silt blocks have decomposed by weathering and erosion from rain and water of some springs near the top of the slide. The slide area, which was previously bare of all vegetation, had meanwhile been partly covered with grass and other herbage, especially on some sand and gravel heaps between the silt blocks. The springs had formed several ponds in the flow area. At the toe of the earth flow, the Rimouski River has considerably eroded the silt blocks and undercut some of them. The silt material originally covering the rock outcrops has been largely washed away into the St. Lawrence River.

In general, the soil strata consist of loose sand and gravel overlying about 20 feet of dense clayey silt. The silt is underlain by a thick bed of fairly insensitive soft silty clay, which generally has a shearing strength between one-quarter and somewhat more than one-half ton per square foot. This clay which has a natural water content of about 30 per cent, plastic limit of 20 and liquid limit of 40 is followed by a dense till on bedrock, mainly clay shale. At the top of the slide, where a fairly deep borehold was made, a thin layer of very dense clayey silt with a high artesian water pressure was found, at a depth of about 20 feet below the surface of the soft clay. At the toe of the slide, disturbed material was found to a depth of about 20 feet and this appears to be the general depth of the slip surface below the present ground level. In the undisturbed soil near the toe of the slide, a small artesian pressure was observed in another borehole.

Causes of the slide

From the evidence obtained in interviews with some 20 eye-witnesses and from the present site investigation, there would appear to be no doubt that the earth flow was caused by erosion of the toe of the bank by the Rimouski River causing a local instability. This caused the first slide, which was probably retrogressive from the River to the upper edge and left the adjacent soil mass in an unstable condition. Thus a second slide took place, which eye-witnesses saw to continue retrogressively for several hours. Stability was not reached until the earth flow extended over the whole width between the stable shale along the northwest edge and the clay and silt along the southeast edge, which had slipped some 100 years ago, and had therefore reached comparative equilibrium.

The earth flow is likely to have been aggravated by artesian groundwater conditions which may have caused a quick condition of the silt immediately after local instability occurred at the toe of the slide. In addition, the soft clay underlying the silt has enabled the silt blocks to slide with little restraint for a considerable distance, causing general instability, as observed by eye-witnesses. A rough stability analysis of the earth flow using the observed approximate slip surface indicates that the average shearing strength of the silty clay was about one-half ton per square foot, which is of the same order as that determined from soil samples.

The relatively shallow depth and flat slope of the earth flow are consistent with other observations on this type of landslide in Quebec and Scandinavia, except that in all these cases very sensitive (quick) clays were involved. The present slide, however, occurred almost exclusively in silt, which is very sensitive to subterranean erosion, as noticed at the River end and other parts of the earth flow.

Conclusions

The field investigation indicates that the earth flow at Rimouski was initiated by erosion of the soil, mainly sand, gravel and silt, at the toe. This erosion caused a retrogressive slide from the River to the upper edge. The earth flow has been aggravated by artesian groundwater conditions and soft clay underlying the silt. The observed depth and slope of the earth flow are similar to those found on other earth flows.

Acknowledgment

The investigation was carried out under the general supervision of Professor J. Hurtubise, Ecole Polytechnique, University of Montreal, on behalf of the Quebec Streams Commission, (Chief Engineer M.J.C. Chagnon) to whom the author is indebted for permission to publish this paper.

References

(1) P. Holmsen "Landslips in Norwegian Quick-Clays". Geotechnique, 1953, vol. 3, p. 187.

(2) C.G. Wenner. "Data on Swedish Landslides". Geol. För. Stockholm, 1951, vol. 73, p. 300.

(3) L.G. Morin. "La Coulee d'Argile de Saint Louis". Nat. Cdn., 1947, vol. 74, p.125.

Fig. 1 Aerial View of Second Slide (August 1951)

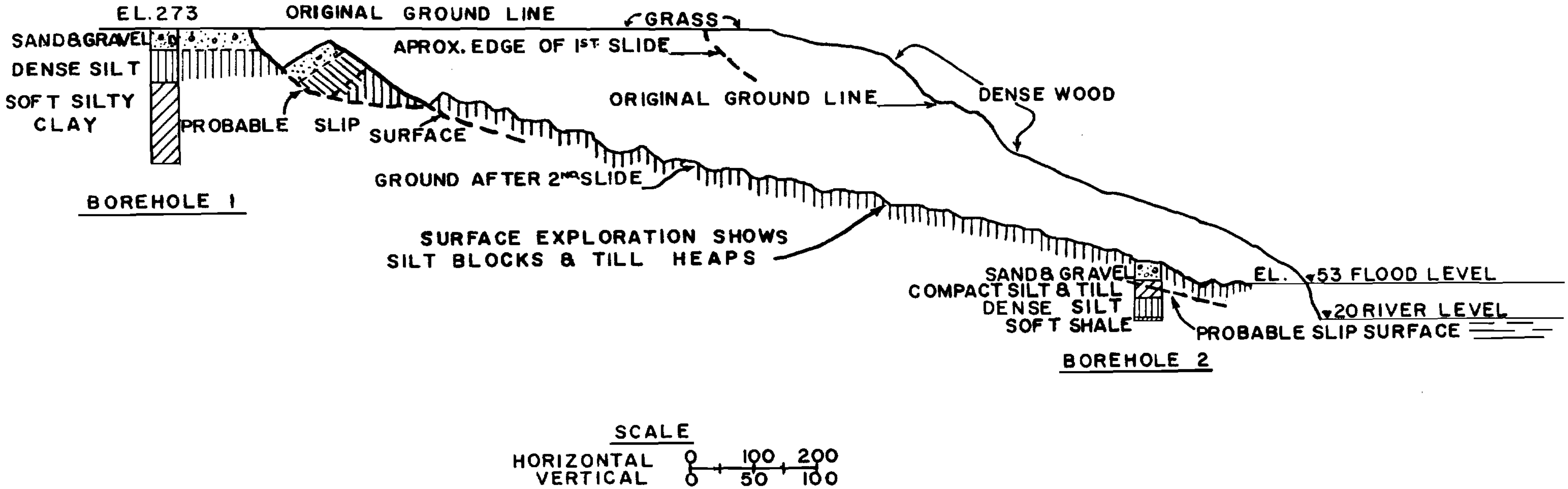

FIG. 2 LONGITUDINAL SECTION OF EARTH FLOW

Discussion

In reply to questions from Mr. Gadd, the author stated that the total height of the slope was about 200 feet. It was not known whether the silt layer and the clay layer were of the same geological origin. The silt was dense and possessed a moderate cohesion, and had a high dry strength. The clay in the slide was soft, relatively insensitive and had a shear strength of 0.2 to 0.5 ton/sq.ft. Regarding the speed with which the slide took place, Dr. Meyerhof stated that the evidence from the eye-witnesses was somewhat contradictory. From the evidence at hand, the first slide took place in approximately 5 minutes. The second slide consisted of a first rush lasting about half an hour, then material continued to drop off the edges for several hours. The actual speeds of the slides may not have been different, as the observers were startled by the first slide.

Dr. McLeod asked if this type of landslip could be analysed by the usual theories of slide failure. Dr. Meyerhof replied that the slide was on a geological scale, and due to several causes, and hence very difficult to analyse. In reply to a question, the author could find no record of unusual heavy rainfalls in the period preceding the slide. Reports of rainfall and other conditions were quite normal.

Mr. Bozozuk asked the author about the causes of the artesian pressures mentioned in the paper.

Dr. Meyerhof replied that the precise causes of pressure were difficult to determine. The investigation took place two years after the slide, and hence much of the evidence had been obliterated.

In response to Dr. Radforth, the author stated that the investigation was done on behalf of the Quebec Streams Commission, with the purpose being to see if it would be possible to predict future landslides of this type. Mr. Ross asked if the trees remained upright during the slide. He reported that he had seen flow slide areas in the Northwest Territories where the trees remained upright after the slide. Dr. Meyerhof said that the trees were knocked over in this slide.

Section 10

Report from the Soil Mechanics Section Division of Building Research

Ground Temperature Project - C. B. Crawford

The study of ground temperatures by the Division of Building Research is following three distinct phases. These are: literature review, field measurements, and laboratory investigations. To date a considerable part of the literature has been reviewed and some field measurements are being made. The laboratory study has been limited to routine classification tests of soils in conjunction with field measurements.

Ground temperature measurements are now being made at several locations in Canada by the Division and in co-operation with other agencies. These locations include Aishihik in the Yukon, Yellowknife and Resolute Bay in the Northwest Territories, Uranium City, Saskatoon, Winnipeg, Labrador, Toronto, and Ottawa. In Toronto the measurements are being made in connection with the new subway. In Ottawa, soil temperatures are being observed at three sites near the laboratories of the Division. In addition to this, records are being kept of frost depth in every excavation made by the City Water Works Department.

The value of the data obtained by the Ottawa Water Works Department has prompted an extension of this aspect of the study. Accordingly, arrangements have been made to collect similar information at Prince George and Nelson in British Columbia; at Edmonton, Saskatoon, and Winnipeg on the Prairies; at North Bay, Ottawa, Montreal, and Quebec City in Ontario and Quebec; and at Fredericton, Halifax, Charlottetown and St. Johns in the Maritimes. These thirteen locations form a nucleus of stations across Canada which could be extended to account for other climatic variations if the first attempt is successful.

In order to simplify the field work, records include only date, location, depth of frost, soil type, and surface cover conditions. It is hoped that these data may be related to air temperature records to assist in the empirical approach to the prediction of frost penetration.

Flat Slab Project - K. N. Burn

The purpose of this study is to arrive at an adequate and economical design for floating concrete slabs constructed on swelling soils.

After much consideration, two concrete slabs were designed and constructed at the Montreal Road site of the National Research Council. The slabs are 20 feet square and 6 inches thick throughout with no perimeter beams nor reinforcing of any kind. One slab was placed on the ground surface from which the vegetation and a minimum amount of soil were removed to level the site; the other was placed on an 18-inch base of well-compacted crushed stone. The concrete was placed in two lifts of 3 inches and on top of the first, heating wires were placed.

Edge insulation was provided to a depth of one foot below the upper surface of each slab. Later the sloping edges of the crushed stone base were covered with sand and sodded to help prevent loss of heat.

Shelter was provided in the form of prefabricated Army huts, and the temperature inside the huts is thermostatically controlled at about 70°F. during the heating season (October 1 to June 1).

Thermocouples buried to a depth of 15 feet below the surface and at various points beneath the slab, at the perimeter and outside, record temperatures in the soil from which the advance and retreat of isotherms are traced. Normally there is sufficient heat loss from the perimeter of each slab in winter to prevent freezing of the adjacent soil.

Vertical fibre pipes 1 1/2 inches in diameter were placed in each slab so that soil samples could be procured when desired by means of auger borings. These pipes are sealed with petrowax against the loss of moisture. After borings are made, the holes are back-filled and sealed. Soil water content is determined in the laboratory and water content profiles are plotted.

Ground movement gauges were also installed beneath each slab at depths of 2 feet and 5 feet below grade. These were placed so that movement could be traced at a number of points from the centre of each slab to a few feet beyond the south edge. To compare the movement of the soil beneath and immediately surrounding each slab with that of natural ground, a number of gauges to indicate movement to a depth of 12 feet 6 inches were installed at the site. The movements of all gauges are read every two or three weeks. In addition to this tracing of ground movement, readings of elevation are taken periodically over the surface of each slab at definite points.

Groundwater table movement is followed at observation wells. The groundwater table varies from the ground surface to about 9 feet below the surface.

The soil is a weathered Leda clay, grey-brown in colour, and highly fissured at the surface. Unweathered clay is encountered at 10 to 12 feet. In exhibits properties of swelling and shrinkage, and in March, 1952, ice lenses were discovered a few inches below the surface. It is believed that the fissures allow sufficient movement of moisture through the clay for the build-up of ice lenses.

Briefly, the results of soil tests are:

Natural water content varies almost uniformly with depth from 30 per cent at the surface to 80 per cent at 15 feet;

Plastic limit is almost constant at 30 per cent;

Liquid limits vary fairly uniformly from 60 per cent at the surface to 80 per cent at 15 feet;

Shrinkage limits are fairly constant at about 25 per cent.

Grain-size analyses show that about 65 per cent of the soil is of clay size and the remaining 35 per cent of silt and sand size, but chiefly in the silt size range.

Because of the fissured nature of the clay, much difficulty was encountered in trimming samples for compression and consolidation tests. Consequently, only a few of these tests were carried out successfully and the results are scattered. Further samples were procured this fall, however, on which these tests will be conducted.

The results of ground movement gauge readings show that nearly all of the swelling takes place in the upper two feet of the soil with the surface change in elevation of the order of 0.2 feet, and that at a depth of 2 feet about .01 to .02 feet. Consequently, the gauges at 2 feet and 5 feet below grade beneath each slab and at the perimeters do not show sufficient movement to differentiate between seasonal variation and that caused by rain storms.

The readings taken over the surface of each slab, however, show that the edges have definitely been raised with respect to the centres. There is considerable difference here between the two slabs. The differential movement between the centre and edges of the slab on the crushed stone base amounts to only about one-quarter of the movement of the slab placed directly on the soil.

It is too early to say whether the design of either slab is satisfactory, but they have both weathered two complete cycles of seasonal changes without any signs of breakage.

Report on Rideau Landslides - M. Bozozuk

The drainage of the Rideau Canal each autumn causes considerable trouble for the Canal Services Branch of the Department of Transport in the way of landslides. Over the past number of years, several landslips have occurred in the high Leda clay banks along the Rideau River between Hog's Back and Black Rapids. One such slide destroyed two summer cottages.

Using the shearing strength of the soil with other factors it is possible to deduce the margin of safety of the bank against sliding along a circular arc with and without changes in river level.

In co-operation with the Canal Services, a slop stability investigation of the Rideau River bank was undertaken at a location considered to be one of the most critical. Three borings on a line perpendicular to the river channel were made:

Boring No. 1 was taken to bedrock and proved by a 10-foot rock core;

Boring No. 2 was taken to refusal in till at a depth of 47 feet;

Boring No. 3, done by hand at the water's edge, taken to 14-foot depth;

Undisturbed samples were obtained as often as conditions permitted using a thin-walled piston sampler.

The borings revealed a non-uniform soil profile with the layers dipping 12° towards the river. The surface layer was relatively loose silty-sand which contributed nothing to the slope stability. Fissured brown silty-clay was found beneath this sand; it,too, yielded unreliable shear strengths. From here to bedrock the profile consisted of alternating layers of silty clay, silty sand, some varved clay, and till. The strength tests were performed on undisturbed samples.

The Fellenius construction was used in the analysis to find the most dangerous circle. It was found that the worst conditions occurred at low water level, with the centre of the sliding circle approximately above a small revetment wall at the water's edge. An attempt was made to find the probably failure surface of a slide which had occurred about a hundred feet away by superimposing the two profiles.

It is possible that, due to the sloping strata, a failure surface would not conform to a circle throughout its length. It may be circular near the top, then parallel to the slope of the soil strata. In fact, trial calculations indicated that the composite failure surface would be more critical than a circular surface.

The best solution to this problem was to flatten the slope and construct a barrier at the toe to stop wave action. Loading the toe with rock ballast, although stabilizing part of the slope, actually aggravates the condition as a new, deeper slide may develop. The rate of river draw-down should also be controlled as a safety measure.

Moisture Studies - E. Penner

The physical properties of soils are largely modified by the water which is held within its framework. Unfortunately, in the past, more emphasis has been placed on the quantity of water involved, than on the force with which it is held. The behaviour of moisture in soils in many aspects is similar to moisture in any other porous material, although there are many moisture problems peculiar only to soils. For those of us who are concerned with soils, an unusual amount of literature on other porous material is at our disposal. In many cases there is the need for intelligent interpretation for subsequent application to soils.

The Building Materials Section of this Division has for some time been actively engaged in thermally activated diffusion studies in porous materials at various moisture tensions. At present the mechanism of moisture movement is being given considerable attention. The Soil Mechanics Section moisture program, although intended to emphasize moisture problems peculiar to soils, will represent only a fraction of the integrated effort by the Division of Building Research on moisture in materials.

Through these projects it is hoped that we can contribute to the understanding of this wide field. Very briefly then we must first study some of the thermodynamic properties of soil moisture. Equipment is being assembled to study the free-energy moisture-content relationships of soils from oven dryness to saturation.

Non-destructive methods of moisture measurement in porous materials is still in a very unsatisfactory state. Two approaches are generally recognized, that is, one can measure the quantity held or the free energy of the moisture. If the free-energy moisture-content relationship is known, the quantity of water held is also indirectly measured.

Commercially available suction meters have been obtained for which electrical-resistance moisture-suction calibrations will be carried out. If improvements in the geometry of the suction meters prove necessary some time may be devoted to this aspect. The most common material used for the blocks is plaster of paris. However, various other materials such as nylon, fiberglas, and porous stones are also used. If it is desired that the moisture content of the soil be known in addition to the suction force with which water is held, the suction-force moisture-content relationship of the particular soil in which the meter is buried must be established. All the necessary data are then available for a graphical solution of moisture content. There are many limitations to this method and although we are not optimistic as to its usefulness in the field, claims for considerable accuracy are being made in recent literature which need substantiation.

An example of a method which measures quantity of water is the neutron scattering method. Plans are being made to measure moisture in this way in connection with the flat slab project.

In addition to empirical studies of frost heaving soils, we expect to be concerned with the more fundamental aspects such as the mechanics involved when moisture moves into the freezing zone, ice lensing phenomena, and the effect of various cations. The problem is recognized as a very large and complex one. Many important contributions have already been made by other institutions.

The Steep Rock Project - W. J. Eden

This is a long-term project undertaken by the Division in its study of northern soils. It deals with varved clays and has the ultimate objective of determining what role the varved structure plays in such properties as shear strength, consolidation, and drainage.

The opportunity for this study arose when Steep Rock Iron Mines were developed. To make mining safe and economical, it was necessary to drain Steep Rock Lake and remove great quantities of the lake bottom deposits by hydraulic dredging. The lake bottom deposits were, for the most part, varved clays. These clays usually have alternate laminations of clayey silt and clay, each about 1/2 inch in thickness.

The first part of the study, almost completed, was devoted to a detailed investigation of the variation in physical properties within the varved couplet, i.e., one dark and one light lamina. Water contents, plasticity, and grain-size tests have been conducted on individual layers and sublayers on a large number of samples. Mineral analyses have been made on a few samples. A few tests have been conducted to ascertain the effect the varved structure has on shear strength.

A paper dealing with this laboratory study is currently under preparation; a summary only of the test results is therefore presented here.

Natural water contents.- A consistent variation of water contents was found between the dark and light layers, as might be expected from the nature of the varved clays. The minimum values occurred in the middle of the light layers, and the maximum in the middle of the dark. The natural water content was found to be almost invariably above the liquid limit, which results in extreme sensitivity to disturbance. The value of water content in the light layers ranged from 20 to 40 per cent, and in the dark from 60 to 100 per cent.

Plasticity.- Variations in both the liquid and plastic limits followed closely the variations in natural water content. In terms of plasticity index the following range in values was found:

light layers	--	3 to 5
dark layers	--	35 to 65

Grain size.- A consistent difference in grain size was found between the dark and light layers, but there was little variation within the layers, as had been the case with water content and plasticity. Clay-sized particles made up from 17 to 35 per cent of the light layers and 65 to 95 per cent of the dark.

Mineral analyses.- The light layers were found to contain quartz, carbonates, feldspar, a little clay, and a trace of organic matter. The dark layers consisted of quartz, feldspar, more clay (probably montmorillonite), and a little organic matter. This analysis was made by Mr. S. A. Forman of the Physical and Crystal Chemistry Section of the Department of Mines and Technical Surveys using the differential thermal and X-ray diffraction techniques.

Shear strength.- As yet, insufficient results have been obtained to draw any definite conclusions. The few tests so far conducted revealed that the orientation of the varves with respect to the direction of stress did influence the shear strength. Maximum shear strengths were obtained when the varves were oriented either perpendicular or parallel to the direction of the major principal stress. Minimum values occurred when the varved planes were inclined at 30 degrees to the direction of stress, with the minimum values roughly one-half that of the maximum. Other tests indicated that the dark layers, in spite of their high water content and high plasticity, were much stronger than the light layers.

Discussion

Mr. Baracos added to the report given by Mr. Burn by mentioning the experimental flat slab in Winnipeg which is a joint project between the Division of Building Research and the University of Manitoba. Dean Macdonald asked if any cracks had appeared in the experimental slabs. Mr. Burn replied that a hairline crack had developed in the slab founded directly on the clay, but it was not considered to affect the slab structure.

Dean Macdonald considered the experimental slabs were operating under ideal conditions and thought if the slab was subjected to the temperature variations and the traffic and loading encountered in the home, the crack might prove more serious. Mr. McRostie asked if there were any reasons for the slabs taking on a dish shape. It was the reverse of what occurred in arid climates where the centre of the slab heaved.

Mr. Baracos reported that the centre of the Winnipeg slab was higher than the edges.

Mr. Schriever stated that the experimental slabs were constructed at the driest time of year, hence any swelling on wetting would be greater at the edges than in the middle. If the slabs were constructed when the soil was saturated probably the reverse would be true.

Mr. McFarlane asked how much difference there was in the movements of the slabs founded on soil and gravel. Mr. Schriever reported that it was in the order of 1/4 to 1/2 inch.

Dr. Meyerhof asked if any triaxial tests, similar to the unconfined results, had been conducted on the varved clay. He suggested that due to the silty nature of the light layers, there may not be so much variation in strength. Mr. Eden replied that to date no triaxial tests had been conducted. Mr. Lea asked how samples were obtained to measure the various properties of the varves.

Mr. Eden replied that it was very tedious work. Special tools had been made to obtain thin slices of the varved samples. Block samples had been used extensively in the testing.

There being no further discussion, the chairman reported that, following the set pattern, the next conference would be held in Ottawa. He would welcome any suggestions on the form of future conferences.

Mr. Baracos mentioned that before Mr. Peterson's talk, a summary of his talk with references was distributed. Mr. Baracos considered this an excellent idea and recommended that it be considered for all papers in the future. Dr. Radforth supported Mr. Baracos' view. Dr. Meyerhof stated that an agenda distributed in advance of the conference would be very convenient. There was general approval of this suggestion.

APPENDIX A

LIST OF THOSE PRESENT AT THE SEVENTH ANNUAL CANADIAN SOIL MECHANICS CONFERENCE

British Columbia

C. F. Ripley, Ripley and Associates, Vancouver

Alberta

R. M. Hardy, University of Alberta, Edmonton

S. R. Sinclair, University of Alberta, Edmonton

Saskatchewan

D. A. Lane, Dept. of Transport, Saskatoon

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

B. B. Torchinsky, University of Saskatchewan, Saskatoon

Manitoba

A. Baracos, University of Manitoba, Winnipeg

A. E. Macdonald, University of Manitoba, Winnipeg

Ontario

J. T. Adams, Hydro-Electric Power Commission, Toronto

J. C. Beauchamp, Dept. of Northern Affairs and National Resources, Ottawa

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

G. E. Bousfield, Defence Research Board, Ottawa

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

P. Brown, Dept. of Public Works, Ottawa

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

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

Ontario (Continued)

L. J. Chapman, Ontario Research Foundation, Toronto

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

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

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

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

J. G. Fyles, Geological Survey of Canada, Ottawa

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

R. C. Gauthier, Dept. of Northern Affairs and National Resources, Ottawa

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

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

D. R. Halstead, Dept. of Public Works, Ottawa

T. A. Harwood, Defence Research Board, Ottawa

E. P. Henderson, Geological Survey of Canada, Ottawa

W. Kalbfleisch, Dept. of Agriculture, Ottawa

K. E. Kofmel, Directorate of Engineer Development, Ottawa

A. Leahey, Dept. of Agriculture, Ottawa

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

J. W. Lucas, Dept. of Public Works, Ottawa

C. L. Merrill, Defence Research Board, Ottawa

A. Michaud, Dept. of Public Works, Ottawa

N. C. Millman, Oshawa, Ontario

J. Morgan, Foundation of Canada Engineering Corp. Ltd., Toronto

R. T. McLaughlin, Engineering Dept., City of Ottawa

N. W. McLeod, Imperial Oil Limited, Toronto

G. C. McRostie, Consulting Engineer, Ottawa

Ontario (Continued)

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

H. C. Nixon, R.C.A.F. Headquarters, Ottawa

J. J. O'Sullivan, Dept. of Public Works, Ottawa

D. Osmun, Dept. of Northern Affairs and National Resources, Ottawa

J. A. Ostrum, Dept. of Transport, Ottawa

R. A. Panter, Ontario Dept. of Highways, Toronto

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

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J. E. Wilson, Armco Drainage and Metal Products Ltd., Toronto

Quebec

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APPENDIX B

NOTES ON VISIT TO THE ROYAL SWEDISH GEOTECHNICAL INSTITUTE

by

N.D. Lea

A little time before the Third International Conference in Zurich, the Royal Swedish Geotechnical Institute held two identical two-day conferences. Dr. Norman McLeod and the writer attended the second conference which was held in the week preceding the Zurich Conference.

On the first day, we visited some construction being carried out by the Department of Public Works in Stockholm, were taken on a tour of the Stockholm Harbour, and visited the Laboratory of the Royal Swedish Geotechnical Institute. On the second day we observed field demonstrations of equipment described below.

LABORATORY

The thing which most impressed me about this Laboratory was the efficiency obtained by having the Laboratory equipment adapted to one size and type of sample container and by the use of automatic equipment. A bank of automatic unconfined compression machines was seen. They are stress controlled and automatically draw a stress-strain diagram, with no further attention from the operator after the sample has been installed.

An automatic consolidation machine has been constructed. One of its notable features is that on the displacement-time diagram, which is drawn automatically, the ordinate is the square root of time rather than arithmetic time.

EQUIPMENT

Equipment seen includes the foil sampler one of which was demonstrated and described at the Fifth Canadian Soil Mechanics Conference. It will therefore not now be described in detail. There have been certain important developments, however, since 1951. Considerable work has been done with rotary drilling in conjunction with the foil sampler and with this procedure it has been possible to obtain continuous samples of almost all soils. The use of drilling fluids in connection with a simple manual frame has also made it possible to obtain samples in sand using this equipment. A new head has been constructed which has certain simplified features - the piston, for example, does not require the continuous holding of the chain.

A pneumatic piston sampler was seen in which the sample tube is pushed into the ground by air pressure acting within the sampler itself. When full penetration is obtained, the air pressure is able to by-pass the sample container and activate shutters, which cut off the sample at the bottom. When the shutter operation is complete, air is injected around the shutters, thus eliminating the vacuum at the bottom of the sample.

A side intake sand and gravel sampler was examined. It consists of a circular steel body with a series of closely spaced holes or chambers drilled from one side. All holes are covered with a steel shutter while the sampler is driven into the ground. When the sampler reaches the required depth, the steel shutter is withdrawn and a feeder and follow-up plate shutters are driven into the same groove. The feeder forces a disturbed sample of soil into the holes in the sampler body. When the sampler is withdrawn, it is emptied on to a board and the result is a small continuous ridge of soil truly representative of the sand and gravel in place. Further details of this tool are given in Proceedings No. 7 of the Royal Swedish Geotechnical Institute.

A vane tester was developed by the Institute in 1947 to 1950. It has been described in detail in Proceedings No. 2 of the Royal Swedish Geotechnical Institute. It has been discussed at previous Canadian Soil Mechanics Conferences. It is operated independently of a borehole. The vane retracts inside a protective cap while the casing and vane rod are driven into the ground.

Certain special piezometers have been developed of which the outstanding feature is that the equipment for measuring the pressure is not left permanently in one location. When it is desired to take a reading, a special cap or insert is lowered through a pipe to a point a few inches above the piezometer tip where it connects by gravity to a valve communicating with the water in the porous stone. The cap or insert is equipped with either a hydraulic or a pneumatic-electric system for measuring the water pressure with no material displacement of water. This device eliminates the frost problem which is serious in other types of piezometers.

Two penetrometers developed by the Royal Swedish Geotechnical Institute were examined; they give promise of being much more economical than the dynamic type.

In the pull-type device a toggle bolt to which a wire rope is fastened is pushed into the ground. When the wire rope is pulled, the toggle opens out. The resistance is measured by a simple manual machine which automatically draws a depth-resistance diagram. Encouraging progress is being made in correlating the resistance with shear strength.

The machine is very sensitive to sand layers in a soft clay deposit. The equipment is designed so that it can be carried on a man's back.

In the rotary penetrometer, the machine is operated by one man who also drives the truck on which it is mounted. The speed of penetration is up to about 10 feet per minute. A point resistance depth diagram is automatically drawn. A slight revision is being made in the penetrometer so that point resistance and frictional resistance will both be auotmatically plotted against depth.

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LIST OF TECHNICAL MEMORANDA (Continued)

24 A suggested classification of muskeg for the engineer. N.W.Radforth. May 1952

25 Soil mechanics papers presented at the Building Research Congress 1951. November 1952

26 Annual report of the Canadian Section of the International Society of Soil Mechanics and Foundation Engineering (June 1951 to June 1952). December 1952

27 Proceedings of the Sixth Canadian Soil Mechanics Conference, Winnipeg, December 15 and 16, 1952. May 1953

28 The use of plant material in the recognition of northern organic terrain characteristics. N. W. Radforth. March 1954

29 Construction and maintenance of roads over peat. F. E. Dryburgh and E. R. McKillop. (Reprinted with permission of D.S.I.R., Great Britain.) July 1954

30 Canadian papers presented at the Third International Conference on Soil Mechanics and Foundation Engineering. July 1954

31 The International Classification for Snow. (Issued by The Commission on Snow and Ice of the International Association of Hydrology.) August 1954

32 Annual Report of the Canadian Section of the International Society of Soil Mechanics and Foundation Engineering. June 1953 to June 1954. July 1954.