MATERIALS

Mr. Mathews

HIGHWAY RESEARCH BOARD

Special Report 60

Soil and Foundation Engineering

in the

Union of Soviet Socialist Republics

National Academy of Sciences—
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HIGHWAY RESEARCH BOARD

Officers and Members of the Executive Committee

1960

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A Report on the Exchange Visit of an American Delegation September 14-October 5, 1959 with The Papers Presented by Visiting Soviet Soil Scientists and Engineers in the United States, May 31-June 21, 1959

Washington, D. C.
1960

PREFACE

From September 14 to October 5, 1959, an American delegation of soil and foundation engineers visited the USSR. The purpose of the trip was to study Soviet research, design, construction and education in the field of soil and foundation engineering.

The American delegation consisted of the following members:

Miles S. Kersten, Professor of Civil Engineering, University of Minnesota, Minneapolis. Chairman of the delegation.

Gerald A. Leonards, Professor of Soil Mechanics, Purdue University, Lafayette, Ind. Secretary of the delegation.

T. William Lambe, Professor of Soil Engineering, Massachusetts Institute of Technology, Cambridge, Mass.

John Lowe, III, Associate Partner, Tippetts-Abbett-McCarthy-Stratton, New York, N. Y.

H. Bolton Seed, Associate Professor of Civil Engineering, University of California, Berkeley.

Gregory P. Tschebotarioff, Professor of Civil Engineering, Princeton University, Princeton, N. J.

Willard J. Turnbull, Chief, Soil Mechanics Division, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Although the length of stay was only 21 days, a large amount of information on Soviet developments in the field of soil and foundation engineering was gathered by the seven members of the delegation. Direct observations were limited to areas around Moscow, Leningrad, Kiev and Stalingrad. However, through reports given by Soviet engineers, and through personal contacts with research workers from other areas, we believe that a coherent picture was obtained of developments in the Soviet Union. Because of the pressure of time during the visit, some errors of fact or interpretation may unknowingly have been included.

The help of the Highway Research Board in administering the arrangements for the visit and the financial assistance provided by the National Science Foundation are gratefully acknowledged by the delegation.

Miles S. Kersten
T. William Lambe
Gerald A. Leonards
John Lowe, III
H. Bolton Seed
Gregory P. Tschebotarioff
Willard J. Turnbull

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I — Introduction

DEVELOPMENT OF USA-USSR EXCHANGE

In August 1957, a Soviet delegation attended the Fourth International Conference on Soil Mechanics and Foundation Engineering held in London, England. After the conclusion of the Lacy-Zarubin Agreement on cultural exchanges between the USA and USSR in 1958, Prof. G. P. Tschebotarioff, who had met the Soviet soil engineers at the London Conference, suggested to Mr. Fred Burggraf, Director of the Highway Research Board, the possibility of arranging an exchange between American and Soviet engineers. The purpose of this exchange was to facilitate an interchange of knowledge in the field of soil engineering, including theoretical and applied research, laboratory and field testing of soils, and design and construction procedures.

After exploring various possibilities, an official invitation from the Highway Research Board and the U. S. Bureau of Public Roads was sent by the State Department to the Soviet Embassy in Washington, D. C., in September 1958. It was hoped that a visit could be arranged that would enable a Soviet group to present papers at the annual meeting of the Highway Research Board in January 1959. This planned exchange could not be carried out for various reasons, but as an outcome of these negotiations, the National Association of Soil Mechanics and Foundation Engineering at the Academy of Construction and Architecture, USSR, proposed to Mr. Burggraf an exchange of small delegations (5-7 men) for "a mutual exchange of experiences in the field of soil mechanics research and its applications."

At the suggestion of Mr. Burggraf the proposed exchange was discussed at the annual meeting of the Highway Research Board in January 1959, and approval to proceed was voted by the Executive Committee. A special committee was appointed by Chairman Harmer E. Davis to work out the plan. The Soil Mechanics and Foundations Division of the American Society of Civil Engineers and the U. S. National Committee of the International Society of Soil Mechanics and Foundation Engineering were invited to join with the Highway Research Board as co-sponsors of the exchange, and these groups accepted.

A proposal to finance the exchange was prepared and submitted to the National Science Foundation in Washington, D. C., and was acted upon favorably. Mr. Joel Orlen, Professional Associate in the Office of International Relations, National Academy of Sciences, was very helpful in all of the negotiations leading to the successful exchange.

A delegation of Soviet engineers visited the United States between May 31 and June 21, 1959. The six members of this delegation were:

Ivan M. Litvinov, Active Member and Secretary, Academy of Construction and Architecture, Ukrainian SSR. Chairman of the delegation.

Nikolai A. Tsytovich, Active Member, Academy of Construction and Architecture, USSR; Professor and Doctor of Technical Sciences; Chairman, National Association of Soil Mechanics and Foundation Engineering, USSR.

Roman A. Tokar, Corresponding Member and Director of the Institute of Foundations, Academy of Construction and Architecture, USSR.

Alexander V. Gladyrevskiy, Chief Specialist of Gosstroy (State Construction Commission), Moscow.

Mikhail M. Levkin, Chief Engineer, Department of Large Bridges, Moscow.

Vassily M. Bezruk, Professor, State Highway Research Institute, Moscow.

The Soviet delegation participated in regional seminars at Princeton, Boston, Urbana, Berkeley, and Washington, D. C., and visited construction jobs in the vicinity of New York, Boston, Chicago, San Francisco and Washington, D. C., as well as the

AASHO test road at Ottawa, Ill. Papers presented by members of the Soviet delegation at these five seminars are published herewith in English translation.

Upon recommendation of the Highway Research Board, in consultation with the American Society of Civil Engineers, President D. W. Bronk of the National Academy of Sciences-National Research Council, approved the appointment of the American delegation to make the return visit to the Soviet Union.

Members of the American delegation received a number of books, reports and pamphlets describing Soviet research projects, design procedures and construction methods—all in the Russian language. These publications, along with those donated during the visit of the Soviet delegation, are listed in Appendix A. They have been filed at the Highway Research Board office in Washington, and are available for use by any interested persons. In addition, Prof. Tschebotarioff has received over a period of time a number of personal copies of Soviet publications. These are listed separately in Appendix A, and provision has been made to furnish microfilms or photostats at cost, or to arrange inter-library loans.

ITINERARY FOR THE DELEGATION

The seven delegates left New York City on September 13 and arrived in Moscow the following day. Three weeks were spent in the Soviet Union in the environs of four principal cities—Moscow, Kiev, Leningrad, and Stalingrad. As was the case for the Soviet visit to the United States, the American delegation had previously requested that the host committee from the Academy of Construction and Architecture, USSR, draw up a preliminary program to include visits to research and design institutes, field construction sites, and educational institutions. This plan worked very well. A few minor changes were suggested by members of the group, mostly to include additional construction sites, and these were incorporated in the final plan.

Transportation, hotels, guides, and the like were also arranged by the Soviet host committee. The delegation was not under the direction of the Soviet Intourist Agency, which resulted in considerable savings in cost and much added convenience. The Soviet host committee, the interpreters, and the engineers in the agencies visited by the American delegation made every effort to insure a successful visit and the visitors were impressed by the preparations which had been made for the technical presentations. Exhibits and photographs of equipment and diagrams for technical descriptions of research studies were exceptionally well prepared. The itinerary follows:

Monday, September 14

4 p.m.	Arrival in Moscow from New York City via Copenhagen.
8 p.m.	Reception dinner with members of host committee, Academy of Construction and Architecture, USSR.

Tuesday, September 15

9 a.m.	Greetings from President N. V. Bekhtin at USSR Academy of Construction and Architecture. Discussion of program.
10 a.m.	Visit to Exhibition of National Economic Developments — Construction Section.
4 p.m.	Visit to Research Institute of Foundation Engineering of USSR Academy of Construction and Architecture. Reports on activities by Soviet engineers and inspection of laboratories.

Wednesday, September 16

10 a.m.	Visit to the laboratories of the Research Sector of "Hydroproject" (Institute for design of hydropower plants). Reports and demonstration of activities by Soviet engineers.

Thursday, September 17

9 a.m. Visit to Moscow Geological Trust. Field demonstration of soil drilling and vibratory soil sampling.

2 p.m. Visit to Institute for Foundation Design of the USSR, Academy of Construction and Architecture. Reports on activities by Soviet engineers.

6 p.m. Visit to field investigation, site for TV tower.

Friday, September 18

9 a.m. Seminar at Moscow University. Tour of building and soil mechanics laboratory between morning and afternoon sessions.

Saturday, September 19

9 a.m. Visits to laboratory of the Research Institute of Frost Studies of the USSR Academy of Sciences.

3 p.m. Visit to construction site of apartment houses.

Sunday, September 20

9 a.m. Visit to Kremlin.

Monday, September 21

10 a.m. Departure for Kiev.

1 p.m. Arrival at Kiev, sight-seeing in city.

8 p.m. Reception dinner, Academy of Construction and Architecture, Ukrainian SSR.

Tuesday, September 22

9 a.m. Visit to Ukrainian Academy of Construction and Architecture. Technical presentations by Ukrainian engineers.

Wednesday, September 23

9 a.m. Visit to Ministry of Construction of the Ukrainian SSR. Discussions of construction and foundation problems for industrial buildings.

12 noon Visit to Kiev Polytechnical Institute.

3 p.m. Visit to Kiev Civil Engineering Institute. Discussion of civil engineering education in Soviet Union and United States.

Thursday, September 24

10 a.m. Seminar at House of Architects. Papers by American delegates.

6 p.m. Continuation of seminar. Discussion and papers by Ukrainian engineers.

Friday, September 25

9 a.m. Sight-seeing.

1 p.m. Departure for Leningrad.

4 p.m. Arrival in Leningrad. Sight-seeing in city.

8 p.m. Reception dinner at House of Scientists.

Saturday, September 26

10 a.m. Visit to construction site of apartment houses.

3 p.m. Leningrad Polytechnic Institute. Laboratory visits, and technical presentations by Prof. V. A. Florin. Presentation of papers by two members of American delegation.

Sunday, September 27

9 a.m. Sight-seeing. Visit to Pushkin and Petrodvoretz (Peterhof).

Monday, September 28

9 a.m. House of Scientists. Presentation of information on vibratory pile driving by Soviet engineers. Visit to field sites for demonstrations of vibratory pile driving.

4 p.m. House of Scientists. Seminar: Presentation of papers by four members of American delegation.

Tuesday, September 29

8 a.m. Departure for Stalingrad.

3 p.m. Arrival at Stalingrad.

Wednesday, September 30

9 a.m. Visit to Stalingrad Hydroelectric Station (dam under construction across Volga River).

Thursday, October 1

9 a.m. Visit to Volga-Don Canal.

1 p.m. Visit to cofferdam construction; vibratory driving of sheet piles.

4 p.m. Visit to pavement construction, hydraulic filling of gullies and apartment-building construction.

6 p.m. Visit to House of Architects. Discussion of master plan for Stalingrad.

Friday, October 2

9 a.m. Sight-seeing.

2 p.m. Departure for Moscow.

6 p.m. Arrival in Moscow.

Saturday, October 3

9 a.m. Visit to highway construction, Moscow area.

3 p.m. Closing discussion at the Academy of Construction and Architecture, USSR.

4 p.m. Visit to soil mechanics laboratories at Moscow Civil Engineering Institute. Presentation of research reports by graduate students.

Sunday, October 4

8 p.m. Farewell dinner.

Monday, October 5

9 a.m. Departure from Moscow.

The Soviet committee also served as hosts for entertainment features whenever time permitted. During the three-week stay members of the delegation attended three ballets, an ice show, two circuses, a puppet show and Cinerama; visited the Kremlin, several museums and art galleries, and were guests at a number of banquets.

In this short report it is not possible to acknowledge all of the Soviet engineers who were so helpful to the members of the American delegation. However, special thanks and acknowledgment are given to Prof. Dr. N. A. Tsytovich, Chairman of the USSR National Association of Soil Mechanics and Foundation Engineering; Director R. A. Tokar, chairman of the Soviet reception committee; Academician I. M. Litvinov, in charge of arrangements at Kiev; Prof. Dr. V. A. Florin, in charge of arrangements at Leningrad; and E. I. Seredinzev who handled arrangements at Stalingrad. The delegation also expresses its appreciation to the many other Soviet engineers who contributed to making the visit so informative and pleasurable.

SEMINARS

Members of the American delegation presented prepared papers at seminars held in Moscow, Kiev and Leningrad. These presentations had a total audience of about 700 persons.

The seminar at Moscow was held at Moscow University. It consisted of two parts and occupied a full day. The first four papers were given at a morning session, from 9 a.m. to 1 p.m. After luncheon a visit was made to the soil mechanics laboratories of Prof. N. V. Ornatsky and to other parts of the Moscow University building. The seminar reassembled at 4 p.m. and continued until 8 o'clock. The following papers were given:

"Studies of Frost Problems in a Northern State," by Prof. Kersten.

"Soil Stabilization," by Prof. Lambe.

"Analysis and Design of Concrete Slab on Ground," by Prof. Leonards.

"Floating Caisson Foundations. Steel Piles with Attachments," by John Lowe, III.

"Recent U.S.A. Research on Soil Strength and Deformation Characteristics Under Dynamic Loading Conditions," by Prof. Seed.

"Certain Features of the Lateral Pressures of Soils," by Prof. Tschebotarioff.

"Critical Elements of Design and Construction of Heavy-Duty Flexible Pavements," by Dr. Turnbull.

Prof. Kersten also displayed a Bureau of Public Roads film on the American Associations of State Highways Officials test road at Ottawa, Illinois, and gave some introductory remarks on the nature and status of the test road.

There were a considerable number of questions on all papers and several of the Soviet engineers gave five-minute talks on the subjects covered. It is understood that the papers given by the American delegates will be published in Moscow in the Russian language.

The seminar at Kiev was held at the House of Architects, and was also a full-day program. The morning session was from 10 a.m. until 2:30 p.m. and the evening program from 6 to 9 p.m. The AASHO Road Test film was again shown by Prof. Kersten. Prof. Leonards, Mr. Lowe, Prof. Tschebotarioff and Prof. Lambe gave papers on the same subjects as listed above for the Moscow seminar. Additional papers, not previously presented, were:

"Foundations for Large Bridges Across San Francisco Bay," by Prof. Seed.

"Summary of Rotary Cone Penetrometer Investigations," by Dr. Turnbull.

Again there were a number of questions on the various papers. At the evening session four of the Soviet engineers presented papers or discussions of soil problems. The papers presented by the Americans will be published in Kiev in the Ukrainian language by the Academy of Construction and Architecture.

At Leningrad the presentations were divided into two parts. Prof. Tschebotarioff presented his paper on lateral pressures and Prof. Lambe discussed "Soil Structure" at the Leningrad Polytechnic Institute on Saturday afternoon, Sept. 26. The second part of the seminar was held at the House of Scientists on Monday afternoon, Sept. 28, and Prof. Kersten first presented his talk on frost problems. He and Prof. Leonards then retired to another room, where a portion of the audience assembled for a two-hour discussion period on frost. The seminar continued with Prof. Seed's talk on dynamic loads, Dr. Turnbull's paper on the cone penetrometer, and one not previously presented: "Current Practice in Soil Sampling in the United States," by Mr. Lowe.

The good attendance of Soviet engineers at the seminars and their stay through the many hours of presentation and discussion evidenced their genuine interest, as did the large number of questions received and answered. Members of the American delegation appreciated the feeling of cordiality demonstrated in these open discussions, and felt that this direct exchange of information was the most effective method of gaining an understanding between the engineers of the two countries.

Nearly all the papers had been translated into Russian before their presentation and could be read directly by one of the Soviet engineers. In one instance a Soviet engineer presented a paper in the Russian language reading directly from the English manuscript. Prof. Tschebotarioff was the only member of the delegation able to present his paper in the Russian language.

Papers presented by members of the American delegation, containing information not currently available in the United States, are published with this report. For the sake of completeness, short summaries of the remaining papers are included.

II — Observations and Impressions

The development of soil engineering in the USSR, its present status and, indeed, its future potential are related to the social and economic "climate" of the country. As a consequence of widespread destruction during World War II, large numbers of structures of all types, especially housing, had to be replaced as rapidly as possible. Enormous manpower losses during the war, superimposed upon an existing scarcity of skilled labor, led to the mechanization of construction processes and to the increased employment of women—as engineers, as technicians, and as skilled and unskilled labor on construction jobs.

The Soviet Union is in a state of rapid economic expansion. This was illustrated by the major construction activities visited by the delegation, which were concentrated on apartment buildings and hydroelectric stations. Large-scale construction of roads and airfields seems to have begun only very recently. Although potential development is obviously great, it is apparent that manpower and material resources have tended to lag behind desired production goals. Accordingly, economy and speed of construction have been emphasized, while quality and aesthetics have been relegated to a role of secondary importance. There is evidence, however, that this emphasis on low initial cost is gradually yielding to considerations of quality.

Central planning of research, design, and construction activities is a basic feature of the Soviet system. Projects of importance to the entire Union are planned and controlled by central administrations located in Moscow. Each of the fifteen republics of the Union has a similar organization for projects of regional importance. Districts and municipalities handle such local matters as housing, streets and secondary roads. Planning, research, design and construction agencies operate as separate entities within the larger organizations.

In its visits to the various academies, institutes, ministries, trusts and construction jobs, the delegation was accorded the greatest respect and cordiality by everyone it encountered. These favorable circumstances were undoubtedly facilitated by the relaxed climate of USSR-USA relations coincident with the visit to America of N. S. Khrushchev, Chairman of the Council of Ministers USSR, which took place during the delegation's stay in the Soviet Union. However, an important factor was the goodwill created by the friendly reception accorded the Soviet delegation during its visit to this country in June 1959. It is felt that the most significant benefit derived from the entire exchange was not the interchange of current knowledge, valuable as this may be, but the personal contacts which, it is believed, will lead to friendly and fruitful exchanges of new developments on a long-term basis.

STATUS OF SOIL ENGINEERING IN USSR

State control and central planning of all activities in the USSR have channeled the development of soil engineering along lines specifically intended to solve the immediate problems associated with the economic development of the country. Heavy demands for maximum economy have engendered a healthy attitude towards large-scale experimentation with new techniques, materials and construction methods. Furthermore, considerable funds appear to be available in support of such work.

Specific accomplishments include the development of precast footings, vibratory driving of piles and hollow caissons, hydraulic filling with sands and clays, thermal stabilization of loess, and construction on permafrost. Extensive data on field performance of structures have been collected and codified into manuals of recommended practice, such as those on the design and treatment of "bases"* and of footings subject to sustained and vibratory loadings. Theoretical treatment of slabs on ground, stress distribution, three-dimensional con-

* Soil layers that support a foundation.

solidation, seepage, and model similarity are exceptionally well developed.

Important projects, such as large dams, canals, locks, and main roads are accorded individual attention. The design of these structures evidence a high level of competence, and, in many instances, ingenious construction methods have been utilized. On the other hand, the quality of finished surfaces and general appearances seem to have been considered of secondary importance, and standardized designs and techniques (particularly for apartment buildings) have not always taken the best advantage of local geologic conditions.

For example, precast reinforced concrete spread footings have been used under soil conditions where large settlements could be expected. Experiments are currently under way on the use of short piles as an alternative to deep footings in situations where the surface soils are highly compressible or the depth of frost penetration is large. The piles require less concrete and are lighter than the equivalent footings.

Soil testing equipment, both field and laboratory, was found to be of generally good design and excellent workmanship. With rare exceptions, all of this equipment was manufactured in the USSR and is of conventional design. Somewhat novel were the torsion shear apparatus and the attachments for conducting direct shear tests under vibratory loading conditions.

Loading systems for consolidation tests generally utilized dead weights with and without lever systems; some oedometers were equipped with pore pressure measuring devices. Triaxial shear apparatus was not plentiful. The delegation did not observe any of advanced design, although drawings were seen for new apparatus that incorporated many of the desirable features now generally recognized. Field plate-loading apparatus of novel design is in use, as are empirical tests for evaluating the danger of liquefaction of sands. These will be described later in the report.

Due perhaps to the general absence of deep deposits of soft plastic clay in the USSR, extensive work on the properties of such soils is not yet apparent; also, the properties of compacted clays and associated field compaction problems appear to have received only minor attention. Work on some of the aspects of cement, bitumen, and chemical stabilization of soils has been only recently activated. While novel methods of soil sampling using vibratory techniques were demonstrated to the delegation, modern equipment for undisturbed sampling of clays was not observed.

EDUCATION AND RESEARCH

Education in general, and especially scientific and engineering education, enjoys a higher level of importance and prestige in the USSR than in the USA. The highly selective system and the large incentives** for scholarly achievement have developed a student body of high caliber and seriousness of purpose that could hardly be excelled anywhere in the world. Undergraduate civil engineering curricula have a broader and more advanced mathematical and scientific base than their counterparts in America. However, at the professional and research level, education in the USSR appears to be more specialized than is the case in most western universities.

The quality of the educational system is reflected in the many highly motivated, dedicated and competent soil engineers and researchers—both men and women—which the members of the U. S. delegation had the pleasure of meeting. In his specialty, the Soviet researcher is apparently completely familiar with work done abroad. A member of the U. S. delegation would often be surprised to find that the Soviet worker with whom he was talking had studied very carefully his latest publications. (The delegation must note with some embarrassment that the reverse of this situation was not true. It is hoped that this exchange may pave the way toward the elimination of this undesirable situation.) The number and quality of scientific and technical personnel being developed are factors whose importance can hardly be overemphasized.

Basic research in the Soviet Union appears to be carried out mainly by the academies of sciences. Research and design in-

** In terms of prestige, higher financial stipend while studying, choice of curriculum and of the type and location of jobs upon graduation.

stitutes concentrate on applied research projects which are, for the most part, directly related to current problems. A considerable amount of applied research is also carried out by the engineering schools, or institutes, as they are called, although basic research also is conducted in them to some extent. Considerable evidence of a gradual broadening of research activities in soil and foundation engineering was apparent in many places.

State support and centralization of research as practiced in the USSR have led to substantial advances in fields which are deemed of particular importance to the economy of the country. The current seven-year plan (1959-65) contemplates a tremendous program of civil works. Ample funds are being provided to sustain a continuous research effort by teams of scientists, engineers and construction specialists. With cleverness and boldness, the Soviet engineers are making every effort to save money in construction and to circumvent the shortage of skilled workers.

They are willing to experiment with new techniques and materials in actual soil engineering projects and apparently have research teams for evaluating these experiments on a scale beyond anything presently contemplated in the United States with the exception of highway construction.

SUMMARY

The American delegation is of the unanimous opinion that the entire exchange was most productive and worthwhile, both in terms of the knowledge gained, as summarized in the remainder of the report, and in terms of potential long-term exchanges of information and experiences.

The status of soil engineering in the USSR is well advanced in areas which, in the past, have been deemed of particular importance to the economic development of the country; these include the development of precast foundation units, vibratory driving of piles and hollow caissons, thermal stabilization of loess, construction on permafrost, etc. However, the delegation saw little evidence of any extensive studies of the shear strength of clays or the properties of compacted soils and field compaction problems; work on cement, bitumen and chemical stabilization had apparently only been recently activated and modern equipment for undisturbed sampling of clays was not observed. As the economy expands, the scope of the Soviet effort probably will be broadened.

Education in soil engineering at both undergraduate and graduate levels is of high quality. Fostered by a highly selective system and by large incentives for scholarly achievement, substantial numbers of highly motivated, dedicated, and competent soil engineers and researchers are being developed.

In anticipation of a large civil works program, funds, facilities and personnel are being provided for long-term research on the performance of actual structures by teams of scientists, engineers and construction specialists. The magnitude of the effort expended on large-scale experimentation and subsequent evaluation appears to exceed that being done in the United States.

Soviet soil and foundation engineers are better informed of work being done in the United States than their American counterparts are of developments in the Soviet Union. The American delegation feels strongly that better methods for obtaining and disseminating Soviet technical knowledge in this field are urgently needed.

III — Academies, Research and Design Institutes

The delegation had the opportunity to visit four research or design institutes in Moscow, the Ukrainian Academy of Construction and Architecture in Kiev, the local branch of the Academy of Construction and Architecture in Leningrad, and the City Trust of Geological Engineering and Mapping in Moscow. At each of these a report of the respective general purposes and activities was first presented, and this was usually followed by technical reports from members of the organization and visits to the laboratories or to field activities.

During these visits the delegation heard many interesting reports and saw numerous pieces of apparatus for laboratory and field testing of soils. A concise review of these experiences follows. In a number of instances, certain types of apparatus and procedures were in use at more than one institute. In such cases, mention of a particular item will be made under the heading of the institute where it was first encountered, or where it appeared to be used most extensively. Thus, the items listed for any one institute should not be construed to comprise all of its activities or facilities.

Institute of Foundation Engineering Research, Academy of Construction and Architecture, Moscow

Dr. R. A. Tokar, director of the Institute, welcomed the delegation. Prof. D. I. Polshin gave a general outline of the Institute's work which is concerned largely with theoretical soil mechanics as this may apply to design and construction problems. Observation on full-scale structures, to evaluate theories, is an important part of the program.

Prof. M. I. Gorbounov-Possadov and colleagues reviewed earlier work and described current studies on the application of theories of elasticity and plasticity to stress distribution and bearing capacity of beams, slabs, and footings on ground. Complete solutions for beams on an elastic half-space, for any loading conditions, are given in Ref. 47, Appendix A. It was stated that solutions had also been obtained for rectangular slabs subjected to a concentrated load (anywhere, except near the edge), or to a symmetrical rectangular loaded area. Warping effects were not considered. Other special loading conditions for which solutions had been obtained also were mentioned.

Work is proceeding on a rigorous solution of the bearing capacity problem based on elastic and plastic zones that differ materially from those of the Prandtl theory. The distribution of contact pressure can also be obtained from the new approach. It is thought that the Terzaghi theory gives values that are too high, and that this will be corrected by the new theory. Preliminary calculations of contact pressures show good agreement with measured values. No reports or papers on the new theory were available at the time of the visit.

The Institute's laboratory contained conventional equipment for routine testing of soils. One project is concerned with possible correlations, on a regional basis, between soil characteristics (Atterberg limits, porosity, etc.) and the engineering properties of sediments.

A section of the laboratory is devoted to research on soil stabilization. The following items were being studied: bituminous stabilization (using emulsions), cement, clay and silicate grouting, soil cement (primarily for building foundations), electrical and electro-chemical stabilization, and stabilization with polymeric materials such as formaldehyde compounds.

Construction jobs where stabilization had been used were described. These included the addition of 7 to 11 per cent portland cement to loess for footings, calcium chloride plus electric current to arrest the movement of slopes, two-stage calcium chloride and sodium silicate injection and injection of sodium silicate alone in loesses already containing salt for foundation strengthening.

Institute for Design of Hydropower Plants, Moscow

V. I. Sevastianov, Chief of the Scientific Research Sector, welcomed the delegation. The Institute is concerned with research and design of large hydroelectric plants, and has laboratories and sections dealing with:

*1. Hydraulic questions related to the design and construction of control structures—including models.
2. Hydraulic machinery — including model turbines.
*3. Foundations and earth construction.
*4. Instrumentation for measurement of stress and strain in structures.
5. Concrete laboratory. (Problems in manufacture of concrete elements.)
6. Field laboratories for measurements on prototype structures.

The hydraulic model laboratory was large and well-equipped. The first model exhibited was of a lock, spillway and powerhouse in the Neman River. The scale was 1:75. Earth embankments exist at each abutment but spillway, powerhouse and lock structure are constructed at some distance out from the abutments. The navigation lock is located at the left end of the spillway structure with the powerhouse beneath it. Flood flows would be passed through the lock.

A hydraulic test on a section of Saratov Dam spillway presently under construction on the Volga between Kuibyshev and Stalingrad was being conducted in a glass flume. The principal problem under study was uplift under the stilling basin. The source of this problem seemed to be wave development up to about 1 meter high. A sill wall at the downstream end of the floor slab was not used for reasons of economy and because it was found that by proper design of the ogee sections a hydraulic jump could be created. The wave, however, caused excessive uplift pressures (up to 6 ft) to develop at the upstream end of the floor slab, thus requiring corrective measures. The measures adopted were (a) a heavier filter, and (b) more weep holes through the slab.

A model of the hydroelectric plant at Saratov was briefly inspected. Vibrational effects of operation were being studied since they were unduly severe. Pulsation pressures were being measured on all elements on which such pulsations might be detrimental. Prototype studies were being made on the Kuibyshev plant. The scale of this model was 1:25.

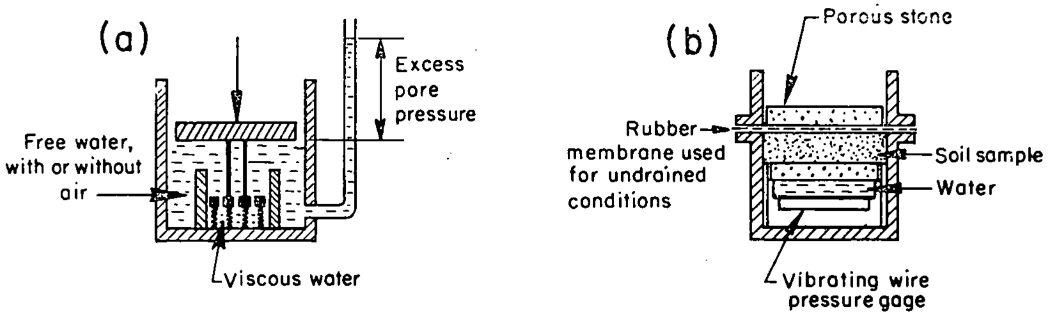
A model of a navigation lock, 1:40 scale, was demonstrated. This lock is in connection with a hydroelectric plant on a tributary of the Volga River. The purpose of the model was to find the minimum time of filling and emptying compatible with other physical features of operation. The model was completely instrumented so that one man could operate it. Instrumentation measured such factors as speed of gate opening, water rise rates, and all physical conditions pertaining to operational features. Directional forces on model barges in the lock chamber were also measured.

A model demonstrating a pneumatic method of investigating the effects of current in fixed and movable bed channels was inspected. In model studies of open channel flow it is necessary to observe the requirements of both the Froude and Reynolds numbers. Consequently, the scale of the model must usually be quite large. If the Froude number is not followed, surface deformations in free flow will develop; the difficulty is avoided with the use of pneumatic pressure models.

The model was constructed to a small scale and covered with an airtight glass plate in order to confine the flow. The small size of the model and rapidity of testing make it a very economical method of investigation. A demonstration was made using burning sawdust. The lines of flow (sparks) were clearly seen and can readily be photographed. Measurements of head losses and other physical parameters are taken. Fine to medium sand can be used in movable bed studies. Heads are duplicated by varying pressure. The model is verified by keeping the ratio of the critical erosive velocity to the actual velocity a constant.

The section dealing with foundations and earth construction had one of the best equipped soil mechanics laboratories visited

* Visited by the delegation.



(a) Model for Concept of Consolidation (b) Pore pressure measurement in consolidation test

FIGURE 1
Model of consolidation process (a) and device for measurement of pore pressures in consolidation test (b).

by the delegation, and a number of active research programs were in progress. The consolidation of saturated and unsaturated clays was being investigated both theoretically and experimentally. The theoretical model under study is shown diagrammatically *(Fig. 1a)*; pore pressures were being measured in the consolidation test with apparatus illustrated *(Fig. 1b)*.

The soil mechanics laboratory at the Hydrotechnical Institute possessed one of the few triaxial compression cells seen by the delegation during the entire visit. It was stated that the apparatus was outmoded, and plans for a new triaxial apparatus were shown. This latter apparatus would accommodate specimens 6 in. in diameter and 15 in. high, apply confining pressures up to 15 kg/cm^2 and axial loads up to 25 kg/cm^2 at deformation rates as slow as 0.03 microns per minute. An automatic null meter for measuring pore pressures *(Fig. 2a)* was exhibited: a field pore pressure meter that could be placed in embankments or on the face of structures was also shown *(Fig. 2b)*. It was stated that measurements of pore pressures in embankments showed that the isochrones do not follow the Terzaghi theory, and the equation

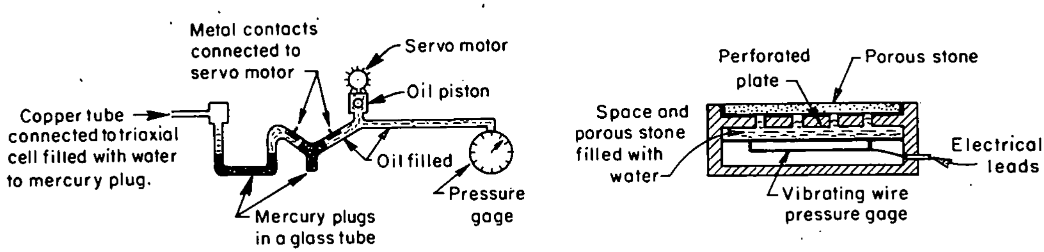
$$\sigma = \sigma' + u + S_t$$

where, σ = total stress
σ' = effective stress
u = pore pressure
S_t = viscous secondary effect

is being used to evaluate effective stresses.

A torsion shear apparatus *(Fig. 3)* is used to evaluate the effects of large strains and to determine the shearing resistance at the boundary between two dissimilar soils. This same apparatus was also seen in a number of other laboratories.

Experiments were being conducted on the liquefaction of sands subject to dynamic



(a) Automatic null meter for pore pressure measurements in laboratory (b) Field pore pressure gage used in embankments and at the face of structures

FIGURE 2
Pore pressure measuring devices.

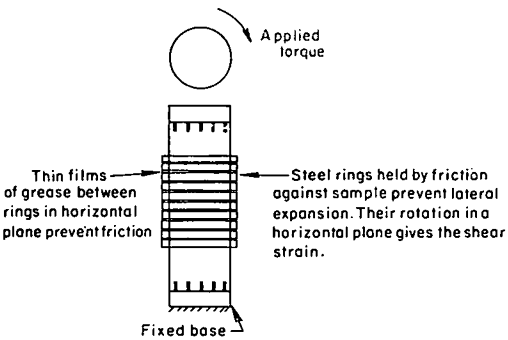


FIGURE 3
Torsion shear apparatus.

loads by measuring the excess pore pressures developed under a variety of conditions. Data were presented to show that the possibility of liquefaction decreases rapidly with increasing confining pressure. Stratification decreases the possibility of over-all liquefaction but localized liquefaction is enhanced.

An interesting aspect of the liquefaction of sands was a study being conducted of the thixotropic properties, exhibited by clayey sands, that were associated with the breakdown of alumino-silicates in the soil by bacteria. Organic matter must be present to enable the bacteria to live under water. To develop thixotropic properties, significant amounts of iron aluminum silicates must be present; when the proportion of silicates reaches about 30 per cent, the sands will exhibit "quicksand" characteristics.

Studies of natural quicksand deposits showed that they were invariably associated with swamps where an ample supply of organic matter was available. The organic matter often took the form of wood-like lignite. After the sands were cleaned by washing, their thixotropic characteristics were greatly reduced.

Model tests on bearing capacity with eccentric and inclined loads were in progress. It was stated that preliminary tests agreed with Meyerhof's theory.

Effects of vibrations in earth dams produced by hydraulic jump in the stilling basin of overflow wiers was being studied with models. It was stated that vertical accelerations as high as 1/10 g could develop. Methods of minimizing the resulting horizontal and vertical displacements of the embankment were being investigated.

Several types of model studies were being conducted on appropriate methods for placing hydraulic fill in a dense condition. One of these models was for the cofferdams being contemplated for use in construction of the high Aswan dam. Stone is first placed under water and the voids are then filled by hydraulic sluicing. The key factor is the grading and percentage of sand in the slurry, which is related to the size of the voids in the rock. Model tests were being conducted to develop design criteria for widely varying conditions. The method had already been used successfully in the field.

The instrumentation laboratory uses SR-4 gages for measuring strains, and vibrating-wire gages for measuring pressures against surfaces and tensions in anchor rods. An electrical analog computer, specifically designed to solve bi-harmonic differential equations, was in use. The biharmonic is broken down into two second order equations, each equation is placed on a separate network, and the computer automatically couples them in the solution. Typical problems being solved are the stresses in a rigid rectangular corner and in rectangular plates on an elastic foundation acted upon by concentrated loads.

Institute of Foundation Design, Academy of Construction and Architecture, Moscow

Director J. Trofimenkov outlined the work of the Institute, where designs for especially difficult foundation problems are worked out. Sections dealing with site investigation, laboratory tests, and field tests provide the basic data for the design section. Close liaison is maintained with the Institute of Foundation Engineering Research.

Reports by the Institute's engineers followed. A typical view of one such session is shown *(Fig. 4)*.

A new vane shear apparatus was described and exhibited. An accurate hole is drilled with rotary tools. A tube with retractable vanes is inserted to the desired depth. Twisting the inner tube causes the

FIGURE 4
Delegation listening to reports of Soviet engineers, Institute of Foundation Design, Moscow.

outer tube to make contact with the sides of the hole and forces the four vanes into the soil. A torque is then applied in the conventional manner.

The economics of short piles, 10 in. square and up to 16 ft long, had been investigated for foundations of 4- to 5-story apartment buildings. Their use has been adopted in cases where the surface soils are compressible, or to minimize excavation (especially in the winter), to avoid the expense of ground water lowering, and where a reduction in dead load of the foundation was desirable.

Foundations subject to dynamic loads were discussed briefly. The delegation was shown a copy of the latest code of practice where tables of allowable bearing pressures for different soil conditions and different classes of machinery are included.

A classification of landslides was presented, and specific examples of slides along rivers, on seashores, and on mountain slopes, together with the remedial measures used, were discussed.

The site of foundation investigations for a prestressed concrete TV tower 500 meters high and weighing 29,000 tons was visited. Load tests were being conducted by a novel procedure. The reaction for a jack is obtained from the passive pressure of the soil on the sides of the pit. By means of a system of cables and pulleys, the deflections of the plate could be read at the ground surface.

Research Institute of Frost Studies, USSR Academy of Sciences, Moscow

Director Prof. N. A. Tsytovich presented the delegation with a two-volume monograph of recently published work performed by the laboratory. It is of interest to note that approximately half of the land surface of the USSR is covered with permafrost.

The laboratory consists of two major sections: (a) a section to investigate physicochemical phenomena in the freezing process, and (b) a section to study the mechanical properties of frozen soils. Emphasis is placed on basic research. Numerous field laboratories to conduct in-situ studies under the direction of the central laboratory are distributed throughout the country.

Dr. S. S. Vialov, assistant director of the laboratory, is working on the properties of ice. Horizontal displacement of ice lenses under foundations have been observed.

Studies are being conducted on the possibility of reducing the rate of frost heaving by replacing divalent ions (such as calcium) in the soil water with monovalent ions (such as sodium). An apparatus has been constructed to determine localized volume changes due to changes in pressure by means of a narrowly focused beam of gamma-rays. Theoretical studies of the shape and movement of glaciers are in progress (see section on Universities and Engineering Institutes).

An extensive investigation to evaluate the stability and deformation characteristics of frozen ground is in progress. The strength-deformation relationships are dependent upon the ice structure in the frozen ground, temperature, soil type and time. Long-term creep studies, with measurements of axial and radial deformations, were being conducted at temperatures as low as —60° C. Properties of frozen sods during and after thaw are also being studied. It was stated that the shearing resistance can drop as much as 80 per cent by alternate freezing and thawing. A "ball-cone" test is used as a measure of strength relaxation with time. The short-term reductions, as measured with a ball-cone, have been correlated with long-term creep tests to give a rapid measure of residual strength.

Some of the future work of the laboratory will be concerned with the problem of artificially freezing soils to assist in the sinking of mine shafts. Design characteristics of soils are to be determined for a shaft 700 meters deep.

Other equipment and studies in the laboratory included:

(a) A 30-ton press in a cold room with temperatures as low as —60° C for studies of creep.

(b) A cold box with temperature to —70° C.

(c) A study of the change in density of soils during thawing and under load. Density was being determined by an apparatus measuring the absorption of gamma rays. It was stated that the sensitivity was 1/100 gr per cc and that the density was measured over a 5 mm depth.

(d) A nucleonic field apparatus for measuring density had been developed, useful to depths of 6 meters or more with a stated precision of 0.025 gr per cc.

(e) A study of the vertical and lateral strains in hollow cylindrical specimens. This work is a start on the study for mine shafts. Mixtures of 3.5 per cent paraffin, 68 per cent sand, and 28.5 per cent mica are being used rather than soil in the preliminary tests.

(f) Studies of the use of ultra-sonic waves for determining the extent of frozen soils either from the ground surface or in bore holes.

City Trust of Geological Engineering and Mapping, Moscow

Chief Engineer Muskalov welcomed the delegation and reviewed the work of the Trust, which is charged with the responsibility of: (a) making plans for the city of Moscow and its suburbs, (b) calculating and laying out boundary lines for buildings and roadways, (c) determining the location of pipe lines and utilities, (d) investigating soil conditions and soil properties, (e) making recommendations concerning desirable types of foundations, (f) examining foundations of existing buildings to determine if additional stories can be added, and (g) investigating causes of reported differential settlement and making recommendations regarding best methods of correction. For this purpose the trust prepares 1:500 and 1:2,000 scale maps and in addition compiles two documents for each building area. The first of these includes:

1. The decision of city authorities concerning the disposition and use of land.

2. A statement of architectural features of the building, which is also decided by the city authorities.

3. The requirements concerning utilities —size of pipes, connections to manholes, etc.

4. A statement concerning type and extent of landscaping to be provided.

5. A brief statement of the geological conditions.

The second document presents the detailed results of the investigation of the soil profile, the results of soil tests, settlement analyses and recommendations concerning the foundations to be used.

For the preparation of these documents the Trust makes about 430,000 lineal feet of boring a year. Almost all of this work is done by the vibratory method of soil exploration—a pipe forced into the ground by a combination of a light static load together with vibrations. Equipment is now available for making borings by this method to a depth of 140 ft although investigations for apartment building foundations usually extend from 25 to 50 ft below the ground surface. The borings are started using a diameter of 5 in. and then reduced to 4 in. or 3½ in. at larger depths. The rate of sinking of the sampling tube is about 1 meter/minute. Allowing time for equipment set-up and sampling, a single boring rig drills between 150 ft and 300 ft per day depending on the type of soils encountered.

Samples obtained by the vibratory boring method are considered to be undisturbed. Initially there was concern that the vibrations would disturb the soil but comparisons of samples obtained by this method with those obtained by driving sampling tubes into the soil show similar results.

A demonstration of vibratory soil sampling was conducted for the delegation. Two sizes of vibrating machines are used depending on the anticipated soil conditions and depth of borings. The smaller unit is held by a truck-mounted crane, weighs 120 kilograms and vibrates at a frequency of 1,400 cycles/min. or some fraction thereof. The desirable frequency is selected during the boring operation. Sampling tubes have a vertical cut along their length, the size varying up to about 1/5 of the circumference of the tube *(Fig. 5)*. This prevents plugging of the soil in the tube during penetration into the ground. The desirable size of cut is selected on a trial and error basis. The vibrator for the larger unit weighs 450 kg., with a power supply of 7 kw.

For deep borings in hard soils, vibratory equipment is unsuitable and percussion drilling is used.

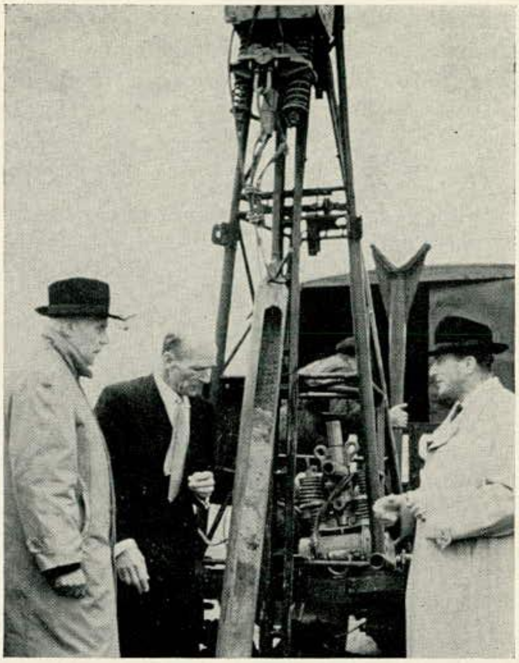


FIGURE 5
Vibratory method of soil sampling.

A machine for drilling test pits with a diameter of 32 in. to a depth of about 30 ft also was demonstrated; this machine can drill at the rate of 6 ft to 8 ft/hour. The entire unit is enclosed in a small truck and is thereby protected against inclement weather.

Ukrainian Academy of Construction and Architecture, Kiev

The delegation was welcomed by President A. M. Komar.* Academician I. M. Litvinov, secretary of the Academy, described its structure and its research and experimental works in soil mechanics and foundations. Formerly a branch of the USSR Academy, the Ukrainian Academy of Architecture was formed separately in 1946 and its activities were broadened to include construction in 1956.

The Academy is now comprised of 14 research institutes, located mainly in Kiev but with small branches in areas of concentrated construction activity. The present staff totals 3,500 of which approximately

* The delegation noted with deep regret the untimely death of President Komar shortly after its return from the USSR.

200 are at the doctoral level, 900 are graduate engineers and scientists and the remainder are technicians. The main problems currently being studied by the Academy are:

1. Methods of reducing the weight of buildings through the development of varitypes of lightweight aggregates.

2. The development of methods for utilization of local materials for construction work.

3. The introduction of new industrial elements (precast and prestressed concrete units) for building constructions.

Problems of mass construction of apartment buildings are now considered to have been solved but the Academy conducts a continuing search for improved construction methods and techniques.

To provide the delegation with a survey of the studies being conducted, the Academy arranged a special exhibition at which representatives of the various institutes and organizations under its jurisdiction presented brief reports on their activities *(Fig. 6)*. Features of the reports are summarized.

A detailed classification of landslides developed in the Ukraine was briefly reviewed.

Problems resulting from differential settlements in areas of mining subsidence were discussed. Of special interest was the use of wedge-shaped footings which were reported to have minimized differential settlements in some areas.

Current studies of the Railway Institute at Dniepropetrovsk were described as follows:

(a) Investigation of the orientation of clay particles due to shear. Studies have been conducted using kaolinite, and photomicrographs show clearly the increase in degree of orientation (parallelism) of the particles along the slip plane.

(b) Development of pore-water pressure gages for use in the field. Three types of piezometers were studied: (1) a counter-pressure pneumatic gage, similar to the Goldbeck cell, (2) a gage

FIGURE 6
Technical presentation at Ukrainian Academy of Construction and Architecture, Kiev.

in which the water pressure deforms a membrane, the deformation being measured with either a vibrating wire or an induction type strain gage, and (3) an improved Casagrande type gage in which the plastic tube was replaced by a copper tube. The latter gage was being favored due to its greater stability for long-term measurements.

(c) Studies of the density and susceptibility to liquefaction of hydraulic sand fills. Hydraulic fills of fine sand are frequently used for dam construction in the USSR, and it has been necessary to determine the danger of liquefaction of these materials. For this purpose different methods of placement have been tried during the construction of small test dams, 15 ft to 30 ft high, and the danger of liquefaction determined by exploding small charges in the dams. It has been found that the density of the hydraulic fill varies considerably depending on the slope of the "beaches" and that utilization of a desirable slope results in a sufficiently dense fill that blasting causes no evidence of liquefaction.

Mobile boring equipment and field laboratories used in the Kiev area were displayed.

The portable soil testing apparatus* developed by Academician Litvinov and used extensively in the Soviet Union was demonstrated *(Fig. 7)*. This equipment, which can be carried in two small suitcases, includes consolidation and direct shear apparatus, an oven, and apparatus for performing classification tests. A paper describing this apparatus was presented by Mr. Litvinov during the visit of the Soviet delegation to the United States. Ten thousand sets of the apparatus have been produced to date.

The method of thermal stabilization of loess soils developed in the Ukraine was described. As about 80 per cent of the area of the Ukraine is covered by loess, there has been considerable research on methods of construction on this type of material. Early attempts at thermal stabilization by injecting hot gases under high pressure in the loess were not effective, but satisfactory results are now obtained by burning gas in a tube in the ground. It has been found that the temperature should not exceed 1,000° C, otherwise the soil grains adjoining the tube will melt and the hot gases cannot penetrate the soil mass. It is necessary, therefore, to provide a proper combination of air and fuel to produce a high temperature but one that will also enable the hot gas to diffuse through the soil.

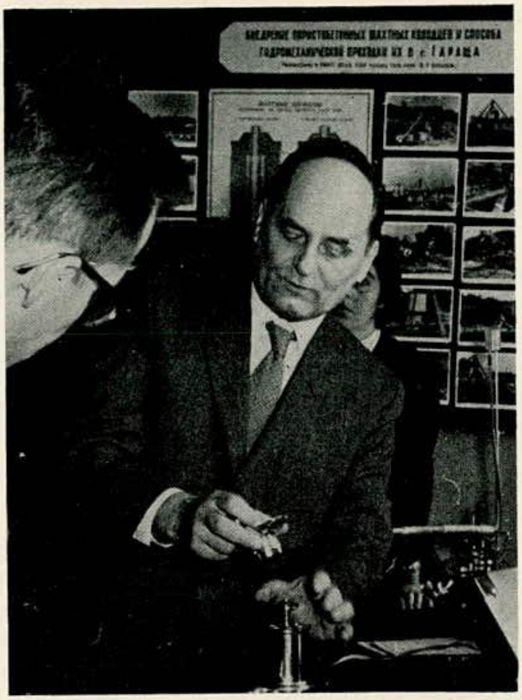


FIGURE 7
Demonstration of portable soil testing apparatus by Academician Litvinov, Kiev.

The present procedure consists of injecting, through pipes in the ground, a controlled mixture of liquid fuel under a pressure of 0.3 atmosphere. The fuel is fired for about 10 days and produces a cylinder of solidified soil, about 9 ft in diameter, around each pipe. The largest depth of the stabilized zone appears to be about 40 ft.

The method has been used to date on about 30 large jobs including foundations for buildings, machinery and smoke stacks. It has also been used for remedial measures in areas where loess has settled due to wetting. The cost of stabilization is about 20

* See "Equipment for Field Geotechnical Investigations of Soils."

rubles per cubic meter of soil, enabling the method to compete economically with pile foundations. However, its use is limited to dry or partially saturated soil.

Pavement design procedures developed by the Auto Road Institute at Kharkov (Director Dr. A. K. Berulia) for use in the Ukraine were described. The design method is based on the principle that a sufficient thickness and quality of base and surfacing must be provided to limit the surface deflection of the pavement to a tolerable amount. This permissible deflection varies from 1 to 1.6 cm depending on the quality of construction.

Computation of the deflection of a given pavement structure is based on an empirical relationship between deflections caused by repeated loads and by static loads and on the deflection computed by elastic theory utilizing a stress distribution pattern determined by applying suitable correction coefficients to the Boussinesq theory. In the Ukraine, moduli of deformation for substitution in the analyses are determined from a correlation obtained from a large number of field studies between the initial composition of a soil and its subsequent modulus of deformation.

Photographs and diagrams illustrating the use of "camouflet" piles were exhibited. The term "camouflet" is used to describe cast-in-place, end bearing piles with an enlarged base produced by exploding a small charge at the bottom of a bored shaft partly filled with fluid concrete.

The method appears to have been first used in England around 1870. The development of the method in the USSR was started by an engineer named Romanov. A hole is made in the ground below the bottom of the pile. The gas of the explosion partly goes up through the concrete, the remainder cools and contracts to form a partial vacuum. The vacuum plus gravity causes the concrete to fall into the hole. The size of the bulb created is indicated by the amount the top of the concrete drops in the pipe. The soil surrounding the tip of the pile is damaged during the explosion.

Camouflet piles have been widely used in the Soviet Union. The greatest depth utilized thus far was for a bridge foundation where the piles were 30-33 meters long. Due to the explosion, the soil in the vicinity of the bulb generally compacts although this is not always the case.

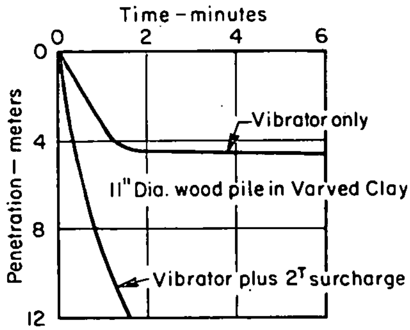
Tests indicate that the unit load which the soil in contact with the bulb can carry is less than the unit load it could carry before the explosion. This decrease in unit load capacity is caused by the cracks which develop during the explosion. However, the large area of the bulb more than compensates for this loss in strength and the capacity of the pile is much greater than if no bulb were used. By using a semi-fluid mortar in the lower portion of the pile, the effective radius of the bulb can be increased about 50 per cent.

Local Branch, Academy of Construction and Architecture, Leningrad

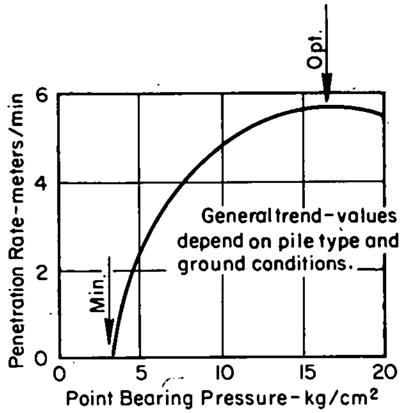
Only one morning was available for a visit to the Academy, which was represented by B. V. Muraviev, vice-director of the Scientific Department. It was devoted to reports on recent developments in vibratory pile driving equipment (for a discussion of applications, see section on Construction Jobs).

Dr. O. A. Savinov reported that the vibratory method of driving piles could be used for all kinds of conditions, but for efficient sinking the parameters had to be properly selected. To overcome point resistance, considerable static loads or impact forces are required, as illustrated *(Fig. 8a)*. Moreover, an optimum amount of surcharge exists for a given set of conditions *(Fig. 8b)*. For high point resistance and heavy piles, as much as 15 tons of surcharge is needed. This involves complicated equipment, although advantage can be taken of the weight of the vibrator and housing.

Vibration at resonance is found to increase efficiency, but requires low frequencies. With vertical vibrations only, there are two instants when the actuating forces are zero (unless two out-of-phase vibrators are used, which decreases efficiency). This permits the soil to set up. It is thought that by applying torsional vibrations 90° out of phase with the vertical vibrations that this difficulty can be cor-

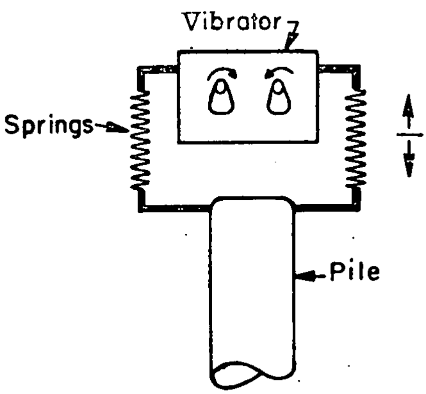


(a) Effect of surcharge on rate of pile penetration

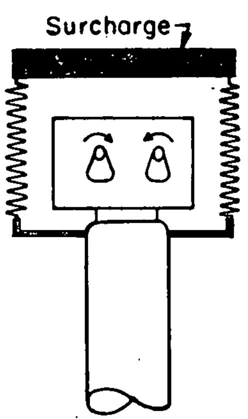


(b) Minimum and optimum point bearing pressures

FIGURE 8
Parameters affecting rate of vibratory pile driving.



(a) Vibrator supported on springs



(b) Vibrator fixed to pile. Surcharge on springs.

FIGURE 9
New designs for vibratory hammers.

rected. Some powerful vibrators based on this principle are now being designed.

Experiments are being conducted with equipment that combines impact with vibration. The successful development of such vibrators would greatly extend their range of applicability. A vibrator of this type is illustrated *(Fig. 9a)*. It was found, however, that it was difficult to control, particularly in the initial stages of driving; did not permit low frequency vibrations; and precluded controlled application of surcharges. Consequently this approach has been abandoned. Experiments are currently under way with a type of vibrator shown schematically *(Fig. 9b)*. This vibrator is attached to the pile and can operate at low frequencies. The magnitude of the surcharge can be varied independently.

The vibratory method of pile driving has also been adapted for other uses, including:

1. Sinking of casing for wells.
2. Vibratory borings and soil sampling.
3. Installation of horizontal conduits without trenching.
4. Mining and earth excavation.
5. Installation of thin-shell caissons.
6. Installation of piezometer pipes.
7. Piercing through frozen soil crusts.
8. Installation of sand drains.

IV — Universities and Engineering Institutes

In the USSR the vital role played by higher education in the military, economic, and social development of the country is apparently fully appreciated at all levels of society; consequently it is treated with the utmost seriousness.

Entrance to the university or engineering institute is considered a privilege accorded only to those who have demonstrated the highest level of academic proficiency. Furthermore, the student's opportunity to select a curriculum of his choosing, his stipend and status as a student, and the possibilities for employment upon graduation are largely determined by academic accomplishments. Accordingly, the eagerness to learn, the desire to excel, and the seriousness of purpose which pervade the Soviet university scene are probably unequaled anywhere in the world.

From the standpoint of efficiency, one cannot help but admire the directness and obvious effectiveness with which the Soviets are going about the business of providing higher education for virtually all of their best minds.

The delegation had the opportunity to visit one university, two polytechnic institutes, and two civil engineering institutes. From these visits, and the ensuing discussions, it is believed that a reasonably clear picture was developed of the Soviet approach to education in civil engineering, and particularly to soil mechanics and foundation engineering. In general, the philosophy is to begin with a strong background in mathematics and the physical sciences, followed by immediate specialization in some aspect of civil engineering, including extensive study of practical design details. A factual account of the conditions encountered by the delegation follows, but no attempt will be made herein to appraise the comparative value of the Soviet educational system.

Moscow University

Moscow University is by far the most impressive building to be constructed in the Soviet Union since the Revolution *(Fig. 10)*. From the 32nd floor, a majestic view is obtained of the surrounding botanical gardens, of the meanders in the Moscow River, and

FIGURE 10
Moscow University.

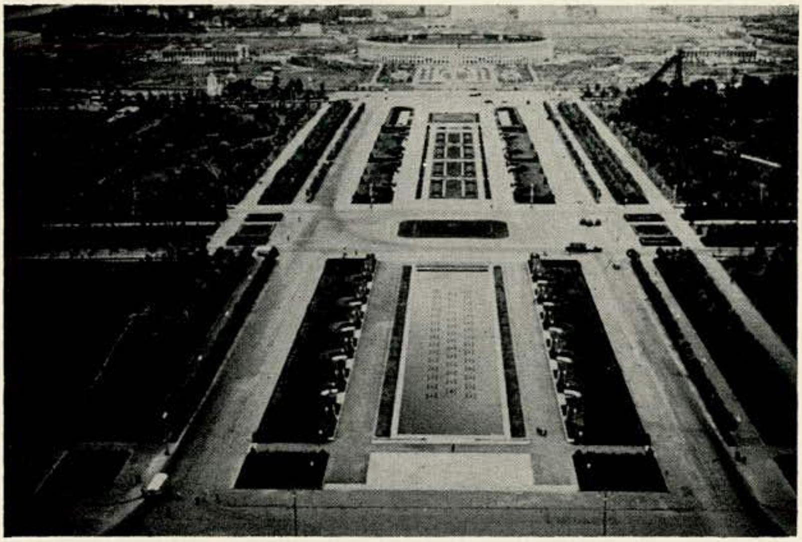


FIGURE 11
View looking north from 32nd floor of the University.

of the city of Moscow *(Fig. 11)*. Mr. Khrushchev's residence is nearby. Muscovites are justly proud of this fine edifice, which is the major seat of learning in the USSR.

The University has an enrollment of approximately 22,000 students of whom about 15,000 are regular day students. About one-half of the students are girls. Natural sciences, consisting of mathematics and mechanics, physics, chemistry, geology, biology and geography are taught in the new building which also houses the museums of anthropology, astronomy, geology and zoology. Approximately 40 per cent of the students are studying the humanities, consisting of languages, literature, history, philosophy, psychology, law and economics, which are taught in old buildings near the center of the city. It is planned to construct a new humanities building in the near future, in order to bring the whole university to one campus.

To enter the University, a student must have a high school diploma and must pass a rigorous, competitive examination. The only exceptions to the examinations are "Gold Medal Students" who enter directly from high school. Only one out of six who take the examinations passes. If a student fails he then goes to the factory or the army for two years. He may then take the examination again, and is given some degree of preference. He may keep trying once a year until he is 35. The average entrance age is 20 years; the limits are 18 and 35.

Ninety-five per cent of the students who start at the University are able to complete the course in due time. Those who do not finish generally drop out for illness or reasons other than lack of ability.

Instruction is free, as in all other Soviet educational establishments. Besides this, the Government provides grants to about 85 per cent of the students. The 15 per cent who do not get fellowships are those who do not need them, e.g., the children of professors or scientists. The fellowship pays for board, room, clothes and similar expenses. Top students get an additional 25 per cent.

Engineering is not taught at the University but the Geology Department includes a Division of Engineering and Hydro-Geology, in which a Chair of Geology and Soil Mechanics is established headed by Prof. N. V. Ornatsky. During intermission between the morning and evening seminars, the delegation made a brief inspection of the geology museum and of the soil mechanics laboratories. The museum contained an outstanding collection of rocks and minerals; the soil mechanics laboratories, used primarily for instruction, contained excellent equipment, all of which was of Soviet design and manufacture.

A direct shear apparatus was fitted with a device which could vibrate the bottom of the shear box horizontally at frequencies ranging between 300 and 3,200 cycles per minute; an unconfined compression device was fitted with a lever along which a weight could be moved by a Selsyn motor, thereby applying continuous loading at a constant rate; otherwise, the equipment was of conventional design. Triaxial compression apparatus was not in evidence.

The general course in soil mechanics is taught in the third year. In addition to the usual material on soil properties (including laboratory experiments) and theoretical soil mechanics, considerable time is devoted to discussing the physico-chemical properties of soils. Elective courses are available in the fourth and fifth years; these consist primarily of special projects, individually assigned.

Moscow Civil Engineering Institute

The Civil Engineering Institute in Moscow prepares students in seven different specialties such as hydraulics, industrial and civil construction, reinforced concrete and sanitary engineering. All students follow a common curriculum for the first two years. Included are 410 hours of mathematics (lectures 200 hours, exercises 210 hours), 133 hours of chemistry (lectures 80 hours, laboratory 53 hours), and 291 hours of physics (lectures 161 hours, laboratory 94 hours, problems 36 hours), as well as courses in freehand drawing and drafting, descriptive geometry, surveying and foreign languages (English, French and German).

Thereafter courses vary with the specialty, but all students receive instruction in theoretical and structural mechanics, strength of materials, theory of elasticity, theory of structures, construction engineering, foundations and soil mechanics.

Instruction in soil mechanics and its applications is provided by the Chair of Soil Mechanics and Foundation Engineering headed by Prof. N. A. Tsytovich, assisted by Dozent P. G. Kouzmin. The course in soil mechanics and foundations is given in the seventh and eighth semesters of the fourth year, and consists of. 98 hours of instruction (lectures 84 hours, laboratory 14 hours) plus a design project.

Each student is given an individual assignment for his project; generally, a geologic cross-section, the results of soil tests, and the general plans for a structure are provided. The student is required to design the foundations, make a settlement estimate, and defend his project before the faculty. About 600 students, divided into 23 separate classes, take the soil mechanics course each year.

After completing a six-year program that includes one year of full-time work in industry, the successful student graduates with the degree of engineer. He must then work at least two years in industry before he can apply for graduate study; if he is accepted, he becomes an aspirant for the candidate degree. Upon completion of one and one-half years of course work he can begin experimental work on his thesis. On the average, another one and one-half years is required to complete the thesis for the candidate degree. Before the thesis is presented to the chair it must first be published either in a book or as a technical paper.

A considerable amount of graduate work in soil mechanics is being conducted at the Institute. Prof. Tsytovich presented five of his aspirants for the candidate degree who described their research projects and answered questions posed by the delegation. They were: Engineer, Mr. Baranov, who was investigating the distribution of contact pressures beneath footings including theoretical and experimental studies of the effects of membrane stiffness on the performance of earth pressure cells; Engineer, Mr. Veronsky, who was developing improved methods for measuring soil density, both from the ground surface and in bore holes, by means of gamma rays; Engineer, Mr. Uskov, who was conducting field investigations on the use of prefabricated reinforced concrete retaining walls constructed at the Kuibyshev dam; Engineer, Miss Doroshkevich, who was investigating the distribution of soil stresses in groups of friction piles subject to static loads—models of pile groups in gelatin are being studied by means of photelasticity; Engineer, Mr. Lukin, who was conducting theoretical and experimental investigations of moisture distribution in partially saturated soils. For analysis, the soil is replaced by a capillary model with an equivalent "suction capacity" (pf). Predictions of the accumulation of moisture under slabs in dry climate are being checked in the field using equipment that measures neutron scattering.

Following the presentation by the five aspirants, Prof. S. S. Vialov briefly discussed the main topic of his forthcoming book which deals with the deformation of ice and the movement of glaciers. Using optical methods, Prof. Vialov has shown that the orientations of ice crystals are aligned in the direction of shear deformations. Measurements of crystal orientations in ice from continental glaciers show alignment that is approximately horizontal; hence it was concluded that shear deformations in such glaciers must also be approximately horizontal. Measurement of stress-strain-time relationships in ice led Vialov to the conclusion that the equation

$$\gamma = a\ \tau^{\alpha}$$

where, γ = rate of change of shear strain with time

τ = applied shear stress

a, α = empirical coefficients that are approximately constant for a given type of ice

adequately describes the shear stress-shear strain rate relationship for ice.

Using these two basic assumptions and a knowledge of precipitation, temperatures, and ground profiles, Vialov has developed a theory to predict the shape of the glacier surface as well as the stress and velocity distributions in the ice mass. Schematic diagrams illustrating his results are shown *(Fig. 12)*. It was stated that the results predicted from this theory were in agreement with field measurements.

Kiev Polytechnic Institute

The Kiev Polytechnic Institute has eleven "faculties" or departments—electrical, radio, heating, machine construction, chemical machinery construction, chemical technology, mining, metallurgy, engineering for the movie industry, evening education faculty, and correspondence education faculty. Within this framework the Institute prepares engineers in 36 specialty areas. Originally the Institute also included a faculty of civil engineering, but because of its size and importance this was separated in 1930 to form the Kiev Civil Engineering Institute.

The present total enrollment of the Polytechnic Institute is 13,000 students. Admission is on the basis of an entrance examination; however, for those students near the boundary line for admission, other factors are considered, such as previous practical experience. For the last academic year, 2,770 students were admitted from about 7,000 applicants.

Under the new plan for engineering education which went into effect in September 1959, students will be educated under a cooperative system combining academic training with practical experience, with four days of study and one day of practical work each week. A half-year of full-time work is also required. To facilitate practical training the Institute has its own factory where a variety of appliances and instruments are manufactured. A practical diploma project must be worked out (problems are assigned by industry) and each student must defend his thesis. Under this system a student will spend six years before graduating with the degree of engineer.

The minimum requirement in mathematics is 300 hours of lectures; the maximum requirement is 450 hours for radio engineers. All students receive 200 hours of instruction in differential and integral calculus, including differential equations. The remainder of the time is spent on topics of particular interest to the various specialty departments, and includes work on the theory of complex variables, operational calculus, vector analysis, partial differential equations and statistics.

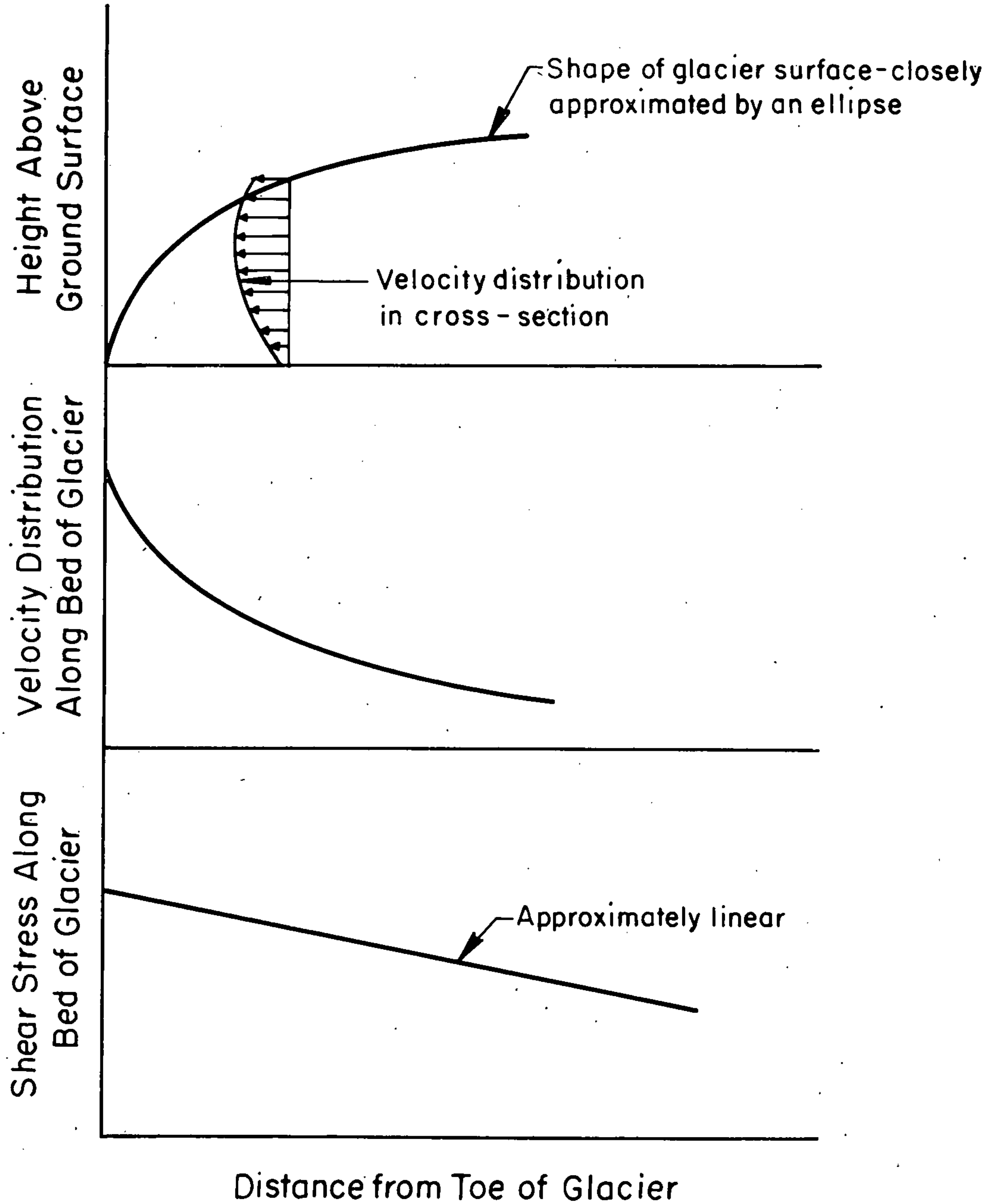


FIGURE 12
Schematic diagrams showing results of Prof. Vialov's theory of glacier movement.

On graduation, students are assigned to positions by a special state commission. A list of available vacancies is sent to the Institute and the professorial staff makes recommendations for the assignment of students based on their performance records. Students disliking their assignments can appeal to the special commission and, if sufficient justification exists, can get the assignment changed. The starting salary of graduates is about 1000 rubles per month.

As at other universities throughout the

USSR, students admitted to the Polytechnic Institute receive considerable financial support. Students from families with incomes less than 500 rubles per month per member of the family receive a monthly stipend of 300 rubles. The better students receive the same stipend regardless of the financial standing of their families, while top students get a 15 per cent bonus. Dormitory space is available for out-of-town students. Typical living costs per student are 220 rubles per month for room and board.

Kiev Civil Engineering Institute

The Civil Engineering Institute also engages in daytime, evening and correspondence instruction. The total student enrollment is 4,500, comprising 2,500 day students and 2,000 in evening and correspondence courses. Areas within which students may specialize include:

- Industrial and Civil Construction
- Sanitary Engineering
  - (a) Water supply and sanitation
  - (b) Heating and ventilation
- City Planning and Construction
  - (a) City economy and organization
  - (b) Engineering geodetics
- Architecture
  - (a) Architectural engineering
  - (b) Interior decoration
- Reinforced Concrete

The same cooperative system of education is followed as at the Polytechnic Institute, but the civil engineering students obtain their practical work at construction sites. During the first two years a student does trade work and is expected to achieve a specified rating in at least three trades. Beyond this stage practical training is more professional in character.

As at the Polytechnic Institute, admission is determined by special examination. On the average there are six applicants for each vacancy, but in certain areas the competition is keener; for example, in the architecture department there are about twelve applicants for each vacancy. About 20 per cent of the total enrollment is female.

Student-faculty ratio in the daytime classes is about 11 to 1 and about 22 to 1 in the evening and correspondence instruction. Daytime students have about 36 contact hours of instruction per week. The total number of contact hours required for graduation is 4,564.

Under the new plan, students will receive 410 hours of instruction in mathematics, 380 hours in physics and 133 hours in chemistry, in addition to technical courses. Each student also will be required to study a foreign language for four years and pass a state examination on completion of the course *(Fig. 13)*.

Soil mechanics instruction at the Institute is provided by the Chair of Foundations headed by Prof. A. M. Drannikov. Two courses, engineering geology and soil mechanics and foundations, are given to students in all departments except heating and ventilation. However, the subject matter is varied somewhat depending on the specialty area of the students.

The course in engineering geology emphasizes the applications of geology in civil engineering construction; it involves 70 to 150 hours of class work depending on the specialty area of the students involved.

The course in soil mechanics and foundations includes lectures, laboratory work and exercise classes. It involves 120 to 190 hours of classwork, with the time being devoted approximately equally to soil mechanics and foundations. Every student is required to complete a series of laboratory assignments, including direct shear and consolidation tests, and an individual design project incorporating some aspect of such topics as strip footings, pile foundations, bridge foundations, anti-seismic design, permafrost, slope stability, quicksand, foundations on fills, etc. The numbers of students run about 50 to 70 for lecture classes, 25 for exercise classes and five to 12 for laboratory classes.

The soil mechanics laboratory was adequately equipped for instructional purposes, although triaxial compression apparatus was not available. Equipment was of Soviet design and manufacture and was of excellent quality.

In addition to student instruction, the soil mechanics staff also engages in research activity. Areas of concentration include: stability of earth masses, including waste

FIGURE 13
U. S. delegation with faculty of Kiev Civil Engineering Institute.

piles and earth fills; economics of precast concrete foundations; utilization of inexpensive explosives for densification of soils; and compaction and lateral pressures of soils along retaining structures. This research is closely related with problems encountered in engineering practice. Subjects for investigation are solicited from practicing engineers, and the research conclusions are tested in practice on completion of the studies.

Leningrad Polytechnic Institute

The Leningrad Polytechnic Institute is one of the distinguished engineering schools in the USSR. It has "faculties" or departments in electrical, mechanical, metallurgical and hydrotechnical fields, and an enrollment of about 4,000 students.

The delegation regrets that an overcrowded schedule did not permit any discussion of curricula. The Soil Mechanics Laboratory, which is headed by Prof. W. A. Florin, is a part of the Hydrotechnical Department whose dean is Prof. P. D. Glebov. The laboratory is apparently used primarily for demonstrations and research.

It is also regretted that time did not permit a visit to the Vedeneev All-Union Research Institute for Hydraulic Engineering (Director, B. V. Proskourakov; Deputy Director, N. N. Maslov), where it is understood significant research is being carried out in soil mechanics, especially in its applications to hydraulic structures. The few available hours were occupied in hearing short reports by Prof. Florin on his current research and a brief visit to his laboratory. Following is a summary of Prof. Florin's discussion:

1. Distribution of Contact Pressure for Footings and Slabs on Ground

It was stated that the assumption of an elastic half-space is to be preferred over Winkler's assumption of a modulus of subgrade reaction if the slab or footing is resting on hard ground, but that Winkler's assumption gives better results in the case of soft, compressible foundations. Solutions have been developed which take into account tangential stresses at the contact surface. It was found that these stresses have little effect on the distribution of normal stresses in the ground, but they do have an important influence on the stresses that develop in the slab or footing.

It is recognized that the use of a visco-elastic half space would be a distinct improvement, and work is proceeding on such analyses.

2. Consolidation of Clays

Prof. Florin referred briefly to his earlier (1937 and 1948) mathematical analyses of two- and three-dimensional consolidation problems. The theory of consolidation needs to be modified to take into account the physical properties of real soils. Among the factors currently being studied are: (a) the influence of entrapped air; (b) the effect of changes in permeability with changes in effective stress, including the variations in permeability due to effective stress gradients; (c) the effect of the compressibility of water and soil particles on the over-all compressibility; and (d) the effect of creep (or secondary compression) during primary consolidation, as well as after primary consolidation is completed.

Prof. Florin is of the opinion that a specific hydraulic gradient is required to initiate the flow of water through clay soils. The results of one particular test were quoted where the magnitude of this initial gradient was about 15 (details of the test such as soil type, degree of saturation, etc., were not given; however, references to work done by S. A. Rosa and B. F. Reltov were supplied). Accordingly, two zones exist in a consolidating mass: in one zone there is flow, and in the other there is not; and the boundary between the two zones is time-dependent. Work is proceeding on mathematical solutions for these conditions.

It was stated that initially a suddenly applied load may be carried entirely by the pore water, or entirely by the soil structure, or partially by both, even in one-dimensional consolidation. Thus, the ratio of the initial excess pore pressure to the applied normal stress may vary between zero and one. Measurements to date to confirm this hypothesis have been confined to partially saturated soils, but the opinion was expressed that it was also valid for saturated soils.

3. Liquefaction of Sands

Reference was made to a slide at a dam site on the Svir River due to liquefaction of sand caused by an ordinary construction blast. At another site, a 5-kilogram charge exploded at a depth of four meters caused a pipeline for hydraulic filling to settle and break. At Gorki Dam a similar explosion resulted in a dish-shaped settlement about 15 meters in diameter, an average settlement of 30 centimeters, and a maximum settlement of 70 centimeters.

The relative density is only one of several factors that influence the susceptibility of a sand to liquefaction: the severity of the vibration, the magnitude of the effective confining pressures, and the shapes of the grains are considered to be of great importance. Susceptibility to liquefaction can be minimized by using surcharges that permit free drainage, as well as by densification. Explosives have been used successfully for the latter purpose. Probes have been inserted in the ground and the changes in conductivity between probes were used as a measure of porosity changes during the explosion. Liquefaction caused by an explosion begins at the surface (where confining pressures are low) and works its way down as more and more overlying material becomes liquefied and loses its confining effects. Densification of the sand, on the other hand, starts from the bottom and progresses upward.

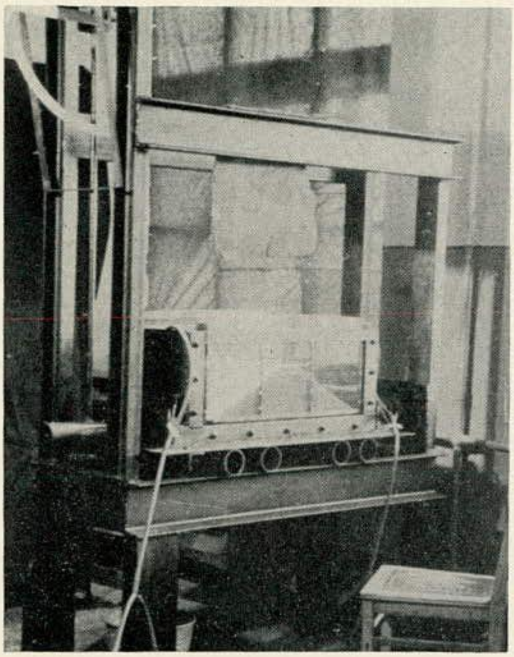
The surface settlement pattern resulting from the detonation of a 5-kg charge at a depth of four meters has been correlated with the susceptibility of sands to liquefaction and design requirements are based on this correlation. However, no details of the correlation were given.

After hearing these reports, the delegation made a brief inspection of Prof. Florin's laboratory. A "quicksand" tank was available that could be rotated on an axis through a measured angle so that slope stability under upward seepage could be demonstrated. A model pile is used to demonstrate electroosmosis. The soil is sufficiently stiff to support the metal pile and a small load. By means of a battery, a difference in potential could be applied between the pile and the container, with the pile being the negative electrode. When the current is switched on excess pore pressures develop around the pile causing it to sink; when the current is turned off the pile comes to rest.

An apparatus to illustrate the effect of varying the normal and shear stresses on the shear strain of granular soils, employing a vibrator mounted on a plate, was shown. The vibrator was capable of applying both normal and shear stresses, which were measured along with the deformations.

Shown is a model embankment *(Fig. 14a)* that can be subjected to shock loading with simultaneous measurements of pore pressure. Typical isochrones of excess pore pressure after 1.5, 3, 6, and 30 seconds are shown *(Fig. 14b)*. Note the negative excess pore pressures near the top of the embankment after three sections.

The delegation was shown a concrete tank (about 10 ft x 10 ft x 10 ft) that had recently been constructed for use in studying bearing capacity under eccentric and inclined loads in both two and three dimensions. The effects of underlying discontinuities will also be investigated.



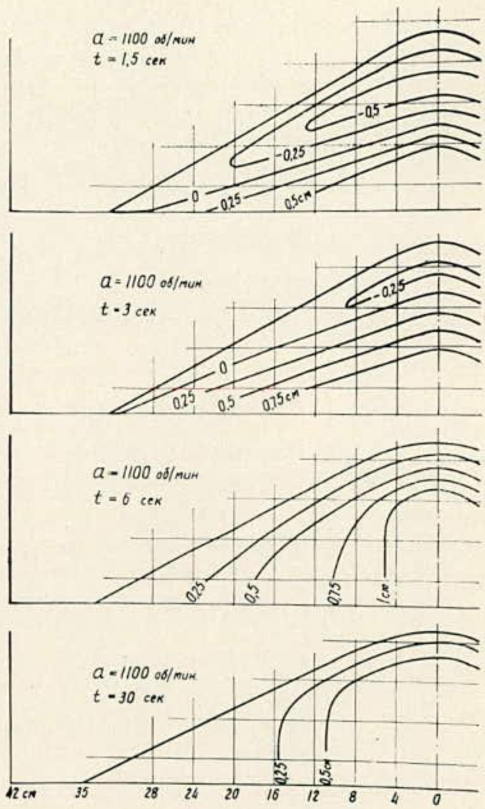


FIGURE 14

Apparatus for measuring excess pore pressures in embankment model caused by shock loading.

V — Construction Jobs

HIGHWAY CONSTRUCTION

The delegation had two opportunities to see highway and pavement construction work: A major inspection of concrete pavement construction and of two highway interchange bridges on the outskirts of Moscow and a small street construction project in Stalingrad where asphaltic concrete was being laid.

It would appear that in past years little emphasis has been placed on highway construction in the Soviet Union. However, with the increasing use of truck transport highways are taking on a status of greater importance, and undoubtedly highway design and construction will be studied and advanced with accelerated efforts in the immediate future.

The visit to highway construction in the Moscow area was conducted by A. M. Sitzky, head of the Moscow Direction of Road Construction, and V. B. Zavadsky, chief engineer of the same organization.

The principal highway system in the vicinity of Moscow consists of 16 main roads radiating out from the city, together with a loop road around the outside of the city, called the Moscow Ring Road. The latter is newly designed.

The delegation first visited a recently finished portland cement concrete pavement on one of the main roads leading out of Moscow, the road to Riazan. The highway has been built on a new location parallel to the old road, which carries a heavy volume of truck traffic. The new highway has a divided section; the pavement slabs are each 7 meters wide, and the dividing strip of grass is 4 meters wide. The slab is 22 cm thick (24 cm is used on some highways). On the outer edge of the slab there is a 30 cm wide strip of white concrete and the outside shoulder, about two meters wide, is surfaced with 12 cm of broken stone. A section of this highway constructed on fill across a marsh has been temporarily paved with stone blocks; it was stated that vertical sand drains had been used here, and that final paving would not be done until the settlement was complete.

Another section of this highway, 800 meters long, is paved with an experimental prestressed concrete slab, 17 cm thick. It was explained that during construction the wires were prestressed the full 800 meters, but that a 2-ft open section was left in the concrete every 100 meters. The wires were cut at these openings after 20 days, and the open sections filled with concrete. Thus the prestressed slabs are 100 meters long. A second experimental section is paved with a 17 cm thick slab of continuously reinforced concrete.

The delegation then visited a field laboratory used for evaluation of highway bases and pavements by means of plate loading tests. The laboratory, which is a wheel-mounted trailer, can apply a static load of 15 tons or a dynamic load of 10 tons to plates of various sizes and record the resulting deflections of the plate and adjacent ground surface. Plate sizes used are: 50 cm diameter for tests directly on the soil, 34 cm diameter for tests on flexible pavements and 34 and 17 cm diameter for tests on concrete pavements.

Seven rods passing through the floor of the laboratory are used for deflection measurements; six of the rods have angles at the ends so that a pattern of deformations can be determined by rotation of the rods.

For dynamic tests, loads are applied to the plates at the rate of 12 per minute and the resulting deflections at seven points are automatically recorded by means of an oscillograph mounted in the laboratory.

Concrete pavement construction was observed on a section of the Moscow Ring Road. A sand base course, 35 cm thick on fill sections and 50 cm thick in cut, was being used; this was compacted by a vibratory compactor traveling on the paving forms.

Density determinations for compaction control were made by a novel procedure. A sample of the base sand of known volume was taken and placed in a chamber of a

FIGURE 15
Density test on compacted subbase.

large hydrometer. By placing the hydrometer in water the technician determined directly the total unit weight of the sample. Next the sample of sand was thoroughly mixed with water, placed in the hydrometer which was now open to the surrounding water and another reading taken. This determination gave the buoyant unit weight of the soil. From these two measurements, and a known specific gravity of solids, the dry unit weight and water content can be calculated. This rapid method would appear to give results accurate enough for field control when soils containing no large particles and possessing little cohesion are encountered.

The pavement slab itself was 24 cm thick and had mesh reinforcement plus two longitudinal bars along each edge and dowels at expansion joints *(Fig. 15)*. The concrete was hauled to the site in trucks without agitators; it was very stiff with a slump of about 1.5 cm. The finishing procedures appeared to be comparatively incomplete; no final longitudinal and horizontal floating was being accomplished. The finishing operation was made more difficult by the ex-

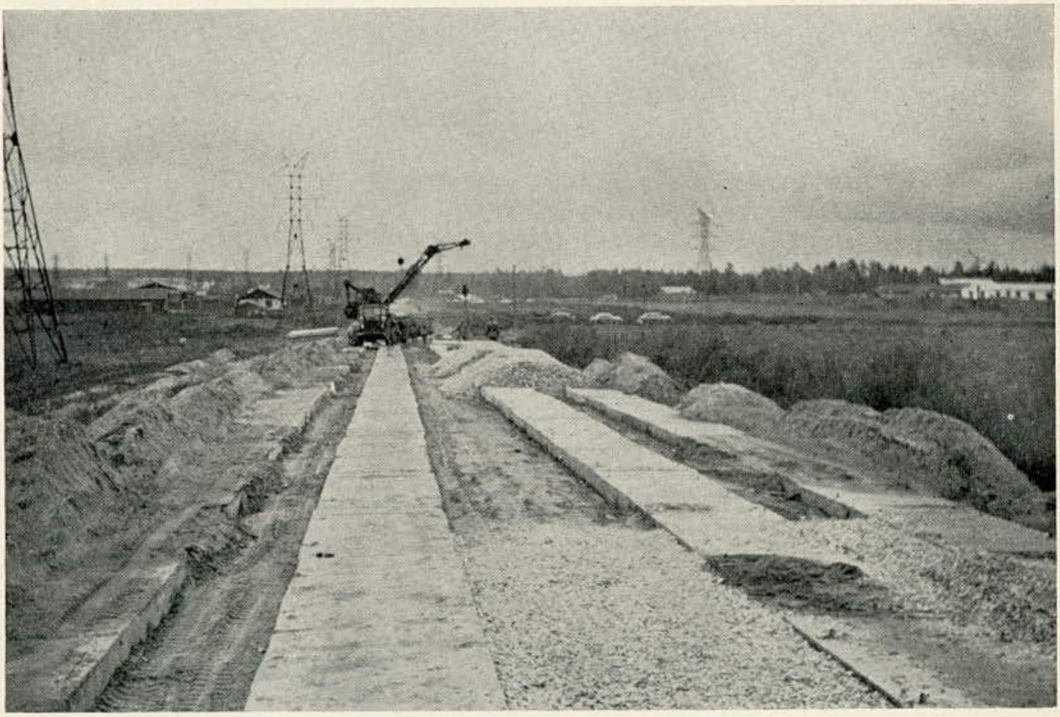


FIGURE 16
Construction of experimental road. Pavement for four wheel tracks has been precast.

tremely dry, harsh mix being used. Curing was by bituminous seal plus 1 cm of sand. Progress was reported to be 220 to 250 meters per day with two shifts.

An interesting feature of the highway inspection was a visit to an experimental track road which was under construction *(Fig. 16)*. The road was being paved with four strips of concrete, each 80 cm wide, and placed to correspond with vehicle wheel tracks. Trials were being made with different types of slabs though all were 18 cm thick. Precast units were about 9 ft long. One portion had the slabs cast in place. Gravel and a light bituminous surfacing was to be placed between the concrete strips. After completion the test road was to have traffic of 4,000 vehicles per day, but it was stated that such construction would be considered for roads with up to 10,000 vehicles per day.

The inspection also included visits to two highway grade separation bridges which were under construction. One of these was the overpass structure carrying the highway to Riazan over another road. The bridge, which was nearly completed, had a good general appearance but on close inspection it showed relatively rough surface finish and roughness at the joints of the precast concrete members. The fill slopes at the spill-through abutments were paved in a neat manner with square precast concrete blocks; other slopes were sodded and presented very good appearance.

The other bridge inspected, crossing the Moscow Ring Road, was being fabricated from precast prestressed units. The delegation saw one of the 22-ton precast segmented members forming a portion of the decking being hoisted into place. The method of post-tensioning these members and grouting the wires was demonstrated on the job, using jacks at both ends of the member. A unique feature of this project was that final grouting of the wires could be continued through the winter thus causing no shut-down of the work; this was accomplished by introducing an electrical current into the wires and maintaining low heat for a sufficient time for the grout to cure.

The only asphalt paving observed by the delegation was on a short section of highway crossing a hydraulically filled gully in Stalingrad *(Fig. 17)*. The section of this pavement consisted of 3½ cm of hot-mix asphalt surface course, 4½ cm of hot-mix asphaltic binder course, 5 cm of slag, 22 cm of crushed rock up to about 8 in. in size, and 5 cm of sand on the compacted subgrade. The asphaltic concrete was being placed and raked by hand. The engineers explained that on larger projects mechani-

FIGURE 17
Construction of asphaltic concrete pavement, Stalingrad.

FIGURE 18
Donwstream view, powerhouse section, Stalingrad Hydrolectric Station.

cal spreading machines are used. A medium-weight three-wheel roller was being used to compact the asphalt producing a reasonably smooth but undulating surface. The side drainage was by curb and catch basin.

STALINGRAD HYDROELECTRIC STATION

The Stalingrad Hydroelectric Station across the Volga River *(Fig. 18)*, just a few miles outside Stalingrad, was the most impressive construction project seen by the delegation in the USSR. The station, construction of which was begun in 1953, consists of a combination of several earth-fill dam sections, concrete overflow gates, a hydroelectric plant, and lock sections.

It is difficult to visualize the structure as an earth dam because of its great crest width and because about 30 per cent of its length is composed of the various concrete hydraulic structures. Beginning at the right abutment, the dam consists of an earth-fill section approximately 1,400 meters long, then the hydroelectric plant and overflow gates with a combined length of about 1,450 meters, then an earth-fill section approximately 900 meters long, then two lock sections occupying a length of about 100 meters and finally an earth-fill section about 1,300 meters long to the left abutment. A channel about 4½ kilometers long with an average depth of 10 to 12 meters, but with a designed minimum depth of 8 meters, has been dredged below the locks.

A harbor above the lock section protects shipping entering the lock. The extension of the left abutment upstream forms one side of this harbor and a dike section about 1,500 meters long forms the other side. It was stated that 10,000-ton ships can pass through the locks; however, the allowable draft was not given. On the basis of observation, it was estimated that a ship with a draft of about 7 meters could be accommodated.

The basic problems in connection with the construction of the dam were: (a) regulation of the discharge of the Volga, (b) production of electrical energy, (c) guaranteed navigation depth for 600 kilometers upstream, and (d) furnishing irrigation water on the Trans-Volga Steppes. The area of the initial irrigation project is one million hectares. The average annual rainfall in the Stalingrad area is about 8 in.

The useful storage capacity of the reservoir is 33 billion cubic meters. The surface area at normal water level is 3,500 square kilometers. The anticipated wave height is 3⅓ meters, which is the reason why the harbor dike mentioned above is required.

The reservoir level was 8 to 9 meters below operation level, but it was intended to completely fill the reservoir in the coming spring.

Foundation. It was difficult to obtain details concerning the foundation materials, but a map in the office of the chief engineer indicated that, basically, the foundation consisted of fine to medium sand 40 to 50 meters in depth. The dry density of the sand was about 94 lb per cu ft, and the dam was constructed directly on this sand without benefit of compaction either on the surface or at depth. In certain areas of the valley the sand was underlain with stiff clay layers at rather shallow depths. Advantage was taken of one of the clay beds in construction of the powerhouse, as the footings of the powerhouse rest in clay.

One particularly interesting feature of the dam was that while consideration may have been given in the design to construction with long piling, all structures were built without piling except the mooring wall adjacent to the locks. This mooring wall is supported on concrete piles. Sheet-pile cutoff sections were used in front of the gate and lock structures; however, it was inferred that these sheet-pile sections did not extend completely through the sand.

Earth Dam Section. The earth-fill portion of the dam was constructed hydraulically; the section ranges from 20 to 40 meters in height and has a crest width of 102 meters. It was understood that this great width of crest was built to accommodate the double-line railroad track, a highway, and appurtenant power-line structures rather than to provide increased stability of the dam.

The approximate downstream slope from the crest downward was 1 on 3½ to 1 on 5, with a flat berm section at the toe of 1 on 18. The upstream slope began at 1 on 3 and graded into 1 on 4, with the slope of the flat shallow berm at the toe being 1 on 18. It was understood that the 1-on-18 slope was the natural hydraulic beaching slope of the sand. Berms were placed on the downstream slope every 8 to 10 meters in vertical elevation.

At maximum operation, as many as six hydraulic dredges were used in constructing the dam. The maximum capacity of each dredge was 1,000 cubic meters of fill per hour. The total volume of material pumped by the dredges was 145 million cubic meters; however, only about half of this was pumped in the dam section proper.

The hydraulic fill material contains very little gravel particles larger than the No. 4 sieve. Delegate Willard Turnbull stated that the sizing in the hydraulic fill material reminded him very much of that in the shells of Sardis Dam, Mississippi.

Apparently the material in borrow was so clean that no trouble was experienced in disposing of the fines. The material was pumped on the fill with the water being allowed to drain laterally over the side slopes and longitudinally between the shoulders of the dam. The slopes were maintained by shear boards.

The closure of the dam was effected in the hydraulic fill section on the right abutment by dumping gravel, stone and concrete tetrahedrons from a pontoon bridge floated across the closure gap. The materials which were dumped were basically of three sizes. The small size, which was dumped first, consisted of coarse gravels and medium-size cobbles; the next-size material was fairly large broken stone; and the dike was finally topped out with heavy tetrahedrons. The first two materials were dumped until the velocity of the current carried the material out about as fast as it was dumped.

After the tetrahedrons were brought above water, the dredges discharged the sand immediately upstream; the sand was forced by the flowing water into the crevices of the stone, and a complete cutoff was effected. It was stated that the closure of the dam was accomplished in one day.

The dry density of the sand in the fill was between 93 and 95 lb per cu yd, which is about the same as that of the foundation material. The problem of liquefaction was considered in the design; however, no danger was indicated.

A figure was not obtained on the amount of settlement under the dam or under any of the hydraulic structures as the result of volume changes in the foundation. However, a statement was made that such settlement as had taken place had in no way been of sufficient magnitude (differentially) to affect the operation of the gates or the eight turbines presently on the line.

It was noted at the abutment of the spillway structure and the hydraulic sand fill that two lines of vibratory driven steel sheet piling extended from the structure abutment into the sand fill as seepage cutoff walls. It was not determined how many of these lines existed across the transverse section of the dam.

Hydroelectric Plant. As indicated previously, the foundation of the power plant rested directly on clay strata. No upstream cutoff was used.

The rating for each turbine is 123,500 kva, 68.2 rpm, 13,800 v, 50 cycles. The capacity of each unit is 600 cubic meters per sec of water at an average head of 20.6 meters, and a maximum head of 26 meters. The total rated capacity of the plant is 2,563,000 kva, which is the largest in the world, followed in order by the Soviet Kuibyshev plant and the Grand Coulee in the United States. The turbine blades, which were built in Leningrad, have variable pitch.

Eight turbines were in operation at the time of the visit, four more were to be completed and cut in on the line later during the year, and the remainder were to be completed and put in operation in 1960. Normally, 12 to 15 units will be in operation at a time.

Two main transmission lines will radiate from the plant, one to the Moscow area at 500,000 v and one to the southwestern regions at 220,000 v. Presently an experimental line of 700,000 to 800,000 v direct current is planned. If this experiment is successful in keeping transmission losses to an acceptable low value, probably all the hydroplants in Siberia will be interconnected, and possibly many more plants in other areas of the Soviet Union will be included.

In addition to the discharge through the turbine, bottom openings below each turbine will accommodate 15,000 cubic meters per sec total. The total discharge capacity of the entire system is approximately 61,000 cubic meters, made up of discharges of 15,000 cubic meters through the bottom gates, 13,000 cubic meters through the turbines, and 33,000 cubic meters through the spillway gates.

Trash grillage protection gates are located in front of the turbines but at a short distance above the intake opening to each turbine rather than directly at the intake. The purpose of this is to minimize head losses.

Many features of construction were observed on the tour of the hydroelectric plant. As indicated, eight turbines were operating; however, the housing over these turbines is temporary, but it will be made permanent when all turbines are installed.

In other areas, early stages of construction were noted. It was observed that the concrete of the main supporting columns for the powerhouse as well as the supporting columns and cantilevers for the railroad and highway had been poured in place. This concrete appeared to have a reasonably good surface finish and was quite free of surface voids and exposed aggregate. Some of the floor slab members were also being poured in place, and placement of reinforcing steel was observed. Many of the connecting girders, beams, and wall slabs were precast concrete. The precast concrete members were inferior in surface finish to the cast-in-place members.

Spillway Gate Structure. The spillway gate structure rests directly on sand. Some of the spillway sections had been completed and the gates were in operation. The spillways are being constructed from the bottom up, in segments, in such manner that flow can be taken over the spillway before it reaches final height. The gates are ordinary slide gates and are operated from an overhead traveling crane. Most of the gates seemed to fit well in that leakage was at a minimum. Each two adjacent gate sections are constructed monolithically. Each monolith is approximately 60 meters wide.

The ordinary flood which has to be accommodated by the plant and system of gates is 40,000 cubic meters per sec, while the anticipated maximum peak flood is about 56,000 cubic meters per sec (10,000-year flood).

Lock Structure. The locks consist of two structures. Each structure has two chambers—the upper chamber upstream and the lower chamber downstream. In other words, the transition in elevation is accomplished in two steps rather than one. The upper lock chamber in each structure rests on sand. Clay and sand were excavated in the area for the lower chamber, which allowed the lower chamber in each structure also to rest directly on sand. Each lock is about 75 ft wide and, as previously stated, will probably pass a boat with a draft of about 7 meters. The mooring wall extends immediately upstream of the lock entrance. The locks are provided with radial gates.

A unique system was used in designing the lock section *(Fig. 19),* which consists basically of heavy concrete walls with half of the base integral with the wall. At the centerline point where the base sections meet an expansion joint is provided with water seals top and bottom. It was understood that no reinforcing exists across this section or transversely between monoliths. At some distance above the base, heavy reinforcing steel connects the two lock walls. This reinforcing steel is securely anchored in each wall but is not encased in concrete.

As the walls are constructed and built up, there is a tendency for greater settlement to take place under each main wall section and a resulting tendency for the wall sections to spread apart. This is resisted by the heavy reinforcing steel of the future lock floor. When a predetermined stress (as determined by its measured elongation) is developed in this reinforcing steel, concrete is placed around the steel, abutting solidly against the inside wall faces. The concrete-encased steel acts as the floor of the lock.

As the steel was in tension when the concrete was placed, the floor is in a sense prestressed. As the backfill is placed behind the concrete walls, the walls tend to tip inward; this force is resisted by the concrete in the floor around the pretensioned steel, which is then placed in compression.

The fine point of the design is that the walls are supposed to tend to come back into the vertical position. There is an open area between the base of the chamber and the floor of the lock, the latter being supported by concrete diaphragms.

Protective Dike to Mooring Basin. The protective dike was constructed by hydraulic sand fill, and at the time the dele-

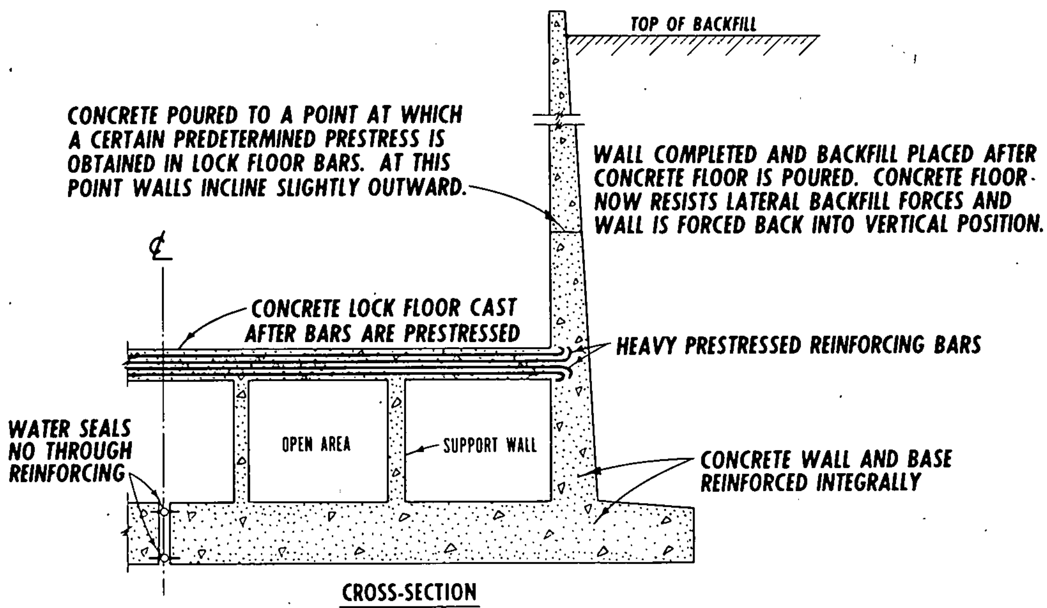


FIGURE 19
Design principle, lock walls, Stalingrad Hydroelectric Station.

FIGURE 20
Hand placement of concrete in protective dike facings, Stalingrad Hydroelectric Station.

gation observed it, the dike was being faced with concrete slabs about 8 in. thick *(Fig. 20)*. The slabs were being placed on a filter bed of gravel. Reinforcing mesh was used in the slabs and preformed expansion joints were constructed around the slab sections, which were about 18 in. by 18 in. The concrete was being placed by hand, and the finished surface was rough and undulating. There seemed to be no continuity of location in placing the slabs on the dike face; rather, it appeared that the completed and unfilled sections were placed in random order. Placement of reinforcing steel and expansion joints, and the alignment of slabs, were not being carefully executed.

Volgski City. An interesting sidelight on the construction of the dam is Volgski City which has been constructed almost in its entirety since 1953. The purpose of the city was to accommodate the workers on the dam. Initially there were about 20,000 workers, and this number gradually grew to 35,000; presently the number is down to about 15,000.

The city is completely self-supporting and independent of Stalingrad and is on the opposite side of the Volga River. It has its own civic and school centers, including libraries, athletic fields, etc. In the libraries it was noted that a few translations of English books were available, among them being books by Jack London, Upton Sinclair, Mitchell Wilson and Mark Twain. One of the conspicuous things about this city was the almost complete absence of passenger automobiles.

When the employment at the dam began to taper off, it was determined that some measures should be taken to provide employment for the inhabitants. As a result, the government began to introduce independent industries into the city. Ten new industries have been introduced and additional people have been brought in to provide a working force for these industries. At the present time the city has a population of 65,000. A general impression concerning the city, and of most of the housing and new buildings being constructed in the USSR, was that the buildings look much older than they actually are.

Impressions. The general impression of the Stalingrad Hydroelectric Station project was that it was of the highest type of construction seen by the delegation in the Soviet Union. In spite of the rather "helter-skelter" appearance, it was understood that construction is approximately on schedule; and it was particularly noticeable that the distribution of personnel about the dam was normal in that there were no unusual concentrations at any one point.

VOLGA-DON CANAL

The Volga-Don Canal, with a minimum water depth of 5 meters, furnishes a waterway for transportation between the Volga and Don Rivers. It is approximately 101 kilometers long, connecting two points which are only about 50 kilometers apart as the crow flies. The designers of the canal utilized natural depressions such as sloughs, channels and lake areas to cut down on the excavation quantities; however, in spite of this, total earthwork was about 194,000,000 cubic meters.

The canal begins at the Tsimlyanskaya reservoir impoundment on the Don River. There are three pump stations between the Don and the high point in the canal between the two rivers, with three pumps at each station, each with a capacity of 45 cubic meters per sec. From the Don River to the high point of the canal there are four locks, representing a total height of about 44 meters. Between the crest and the Volga River there is a drop of about 88 meters through nine locks. The Volga-Don Canal is a very important link in the waterways system of the USSR.

The soil profile along the line of the canal consists of diverse soil types of the Quaternary period. Some of the soils are relatively watertight, while some are pervious. The leakage tends to be offset by relatively high water tables.

Some of the major features considered in the development of the canal were navigation, fisheries, irrigation and power. The power is developed in one plant which has a capacity of 160,000 kva.

Lock No. 2. The delegation visited Lock No. 2, which is typical of the several locks in the system. Eight minutes is required to fill the lock, which is 18 meters wide and 500 ft long. The lock is equipped with radial gates and is capable of transporting 3,000-ton barges with a 15-foot draft. It is intended to increase the barge capacity to 5,000 tons. Materials transported are lumber, coal, oil, oil products, and various types of grain. Passenger ferryboats also use the system. The lockage time ranges from 12 to 15 minutes, depending on the type of boats.

In the excavation for most of the locks, ground-water lowering was required. This was accomplished either by shallow wellpoints or deep wells. The lock is constructed in seven longitudinal sections with nothing in the expansion joints between sections other than water seals.

Each longitudinal section is divided along the centerline of the lock into two monolithic sections. The monoliths rest directly on the soil without pile support. The centerline section between the two monoliths is not reinforced—only water seals are incorporated in it.

It was interesting to note that this relatively simple foundation is performing satisfactorily. About the only detrimental effect noticed around the lock system was a slight slumping of the outer slopes of the backfill behind the lock wall chambers. Some seepage flow was noticed in the ditches at the toe of these slopes.

Canal. The canal has a bottom width of 38 meters and side slopes from bottom upward of 1 on 6 to 1 on 3 at the top. In cut sections no sealing of the canal is used in any of the soil types. In the more pervious soils in the cut-fill sections, a clay blanket approximately 2 ft thick was used. It is considered that the main losses from the canal are due to evaporation because of the generally high water table. In more erodible materials the sides of the canal are protected by rubble stone and at some points by concrete revetments resting on a three-layer filter blanket.

DRIVING OF BEARING AND SHEET PILES AND CAISSON SHELLS

The first reports concerning Soviet vibratory driving techniques attracted considerable attention in the United States. As a result, an impression was created in some quarters that vibratory driving was the only one used in the Soviet Union. No statements to this effect were either made or implied by any Soviet engineer during the entire exchange. Pile driving equipment seen by the delegation on various construction jobs included several types of vibratory drivers as well as drop hammers and single-acting and double-acting steam hammers.

The various types of vibratory hammers in use in the Soviet Union, as well as those under further development, have been described by a member of the Soviet Delegation.* The delegation was taken to see three construction sites where vibratory pile driving was in progress.

The first vibratory driving job visited was in Leningrad, where U-shaped steel sheet piles were being driven for the excavation of a sewer trench on a new housing project. A VP-2 type vibrator was used for the purpose. The ground appeared to be soft and the vibratory driving proceeded easily.

Next, the delegation visited an old pier in Leningrad Harbor which was being re-

* See "The Use of the Vibratory Method for Sinking Piles and Pile Shells in Bridge Construction of the USSR" by M. M. Levkin.

habilitated by driving in front of it a row of U-shaped steel sheet piles and a row of precast reinforced concrete piles which were to support a relieving platform.

Most of the piles and sheet piles had been driven by means of a VP-2 type vibrator. This vibrator develops a centrifugal force of 9.2 tons and is designed to drive piles with not more than 50 tons limit resistance. At one end of the pier, a layer of stiff clay was encountered which could not be penetrated by this type of vibrator. A 6-ton single acting steam hammer was then employed, but not all the precast concrete piles could be driven down to grade as was evidenced by their condition in place at the time of the visit. The heads of the steel sheet piles driven by the 6-ton hammer evidenced considerable distortions.

In spite of these unfavorable conditions, an attempt was made to demonstrate the action of the VP-2 vibrator available on the site by driving a U-shaped steel sheet pile specially set up for the purpose. This sheet pile, however, could be driven down only about half way.

On the same site a large unit was exhibited consisting of two synchronized VP-160 vibrators which were stated to have been used to drive precast reinforced concrete cylindrical shell dolphins in another part of the Leningrad Harbor *(Fig. 21)*. Vibrators of this type are said to develop up to 320 tons downward force, and are now used for sinking caissons for bridge foundations consisting of cylindrical precast shells up to 18 ft diameter.

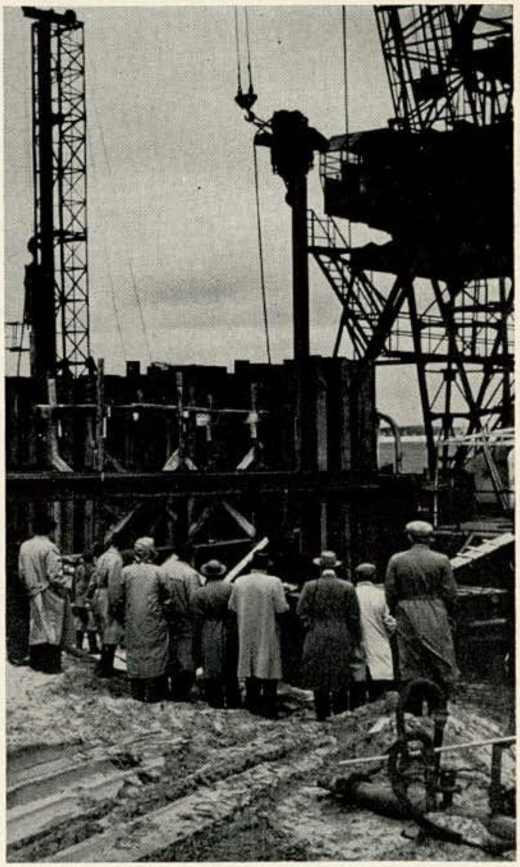


FIGURE 22
VP-2 vibrator driving sheet piling. Cofferdam construction on Volga River, Stalingrad.

FIGURE 21
Unit with VP-160 vibrators for driving reinforced concrete cylindrical shell dolphins, Leningrad Harbor.

In the Stalingrad area, the delegation visited a site where a double-walled steel sheet pile cofferdam for the construction of a landing pier was being installed *(Fig. 22)*. A VP-2 vibratory driver was used most of the time but a double-acting steam hammer finished the driving when the pile resistance exceeded the capacity of the VP-2 vibrator. The river portions of the cofferdam were already completed at the time of the visit and the wing sections were being installed to make a junction with the Volga River banks. These banks had old protection mattresses below their surface.

The delegation observed a 68-ft U-shaped steel pile driven down to grade by a VP-2 vibrator. About half way down it encountered some obstruction and the penetration was slowed appreciably. Vibratory driving was then interrupted for about four minutes every three minutes in order not to overheat the motor. The delegation was informed that the vibrator was not designed to run longer, since in sandy soils with no obstructions it took about 2½ minutes to sink a sheet pile of that length.

Vibratory driving of sheet piles is of particular advantage in dense saturated sands and gravels. The organization for construction work in the Soviet Union is on a regional basis, which facilitates extensive use of vibratory drivers by the same construction thrust whenever favorable soil conditions are encountered.

APARTMENT BUILDING CONSTRUCTION

One of the main efforts in building construction in the Soviet Union today is in the field of housing. The concerted activity on apartment building construction was very obvious in all the cities visited, and the delegation inspected construction sites in Moscow, Leningrad and Stalingrad.

In order to provide some idea of the magnitude of this construction program, the delegation was informed that 90,000 flats were being built in Moscow in 1959, providing about 2,800,000 square meters of "living area" (i.e., living and bed rooms). Kitchens, bathrooms, hallways, and stairways are additional to this area, and would increase it by about 30 per cent.

In Leningrad, it is planned to almost double the existing housing by 1965; in 1959, about 600,000 square meters of living space was being produced. In Stalingrad which now has a population of 600,000 and has about 3,5 million square meters of living space, present plans are for a growth to 800,000 persons, with living space to be increased to 9 or 10 million square meters.

Most of the apartment buildings which the delegation saw under construction were five-story structures. The buildings were about 10 meters wide and 64 meters long and were of wall-bearing design. There are 60 flats in such a building and four stairways; some buildings have 80 flats. Most flats consist of two or three rooms plus kitchen and bathroom with about 35 square meters of "living space". Living space is allocated at about 9 square meters per person. Utilities such as gas, electricity, water and central heating are provided.

The field construction of these apartment buildings is essentially an assembly process. Precast reinforced concrete footings, wall panels, floors, stairways, bathrooms, etc., are made up in plants, hauled to the sites, and hoisted into place with a tower crane. Connections between the panels are made by welding. Pictured is a view of this construction in Moscow *(Fig. 23)*.

Plants which fabricate wall panels were visited in Moscow and Leningrad. The Leningrad plant was one of five in that city

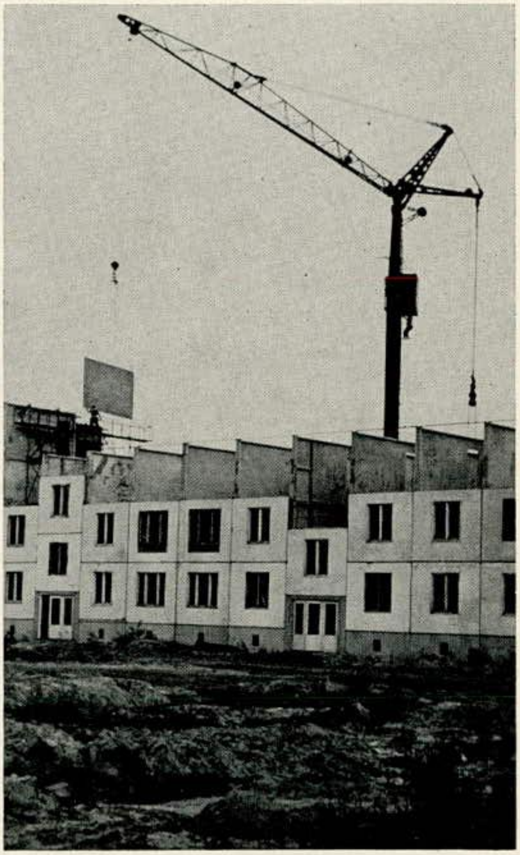


FIGURE 23
Apartment building construction, Moscow. Precast units being placed by tower crane.

each of which produces all of the panels for a particular apartment house design with the exception of the footings. The plant visited has a capacity for producing enough living space for two 80-flat buildings per month. Another plant in Leningrad was reported to have a capacity of 170,000 square meters per year.

Concrete panels produced at the plant were steam-cured for 12 hours and those for exterior walls were provided with a foam concrete insulation. They were then finished to a point of maximum readiness for incorporation into the apartment structure; windows and doors were included in wall panels and plumbing was installed in the bathroom units before delivery to the construction site. As a result of this high degree of prefabrication, an apartment building with 80 flats is constructed in 25 working days (three shifts per day) after completion of the foundation; the building is completely finished in 3½ months.

The delegation was particularly interested in the foundation aspects of these structures and in the methods of designing for settlement. Individual footings for columns and strip footings for the bearing walls were all of precast concrete *(Fig. 24)*. The size of these varied according to the bearing value of the soil. In Moscow it did not appear that there had been any special treatment of the soil before the footings were placed. In Leningrad the foundation conditions are generally quite poor and deposits of organic silt or peat are frequently encountered.

In the past some housing units have been constructed over such poor soil deposits and have settled as much as 40 centimeters resulting in considerable damage. Consequently efforts are now made to conduct more careful foundation investigations and to provide adequate foundations. It was stated that preliminary borings are made at each building site at spacings of 50 to 150 meters and to depths of about 10 meters. Where necessary, pile foundations are being used.

Another measure taken with the object of minimizing differential settlements is the provision of a ring of cast-in-place reinforced concrete just above the foundation wall blocks and below the first floor. The ring has the same width as the foundation wall and is about a foot thick; this was the only poured concrete observed by the delegation during the several visits to apart-

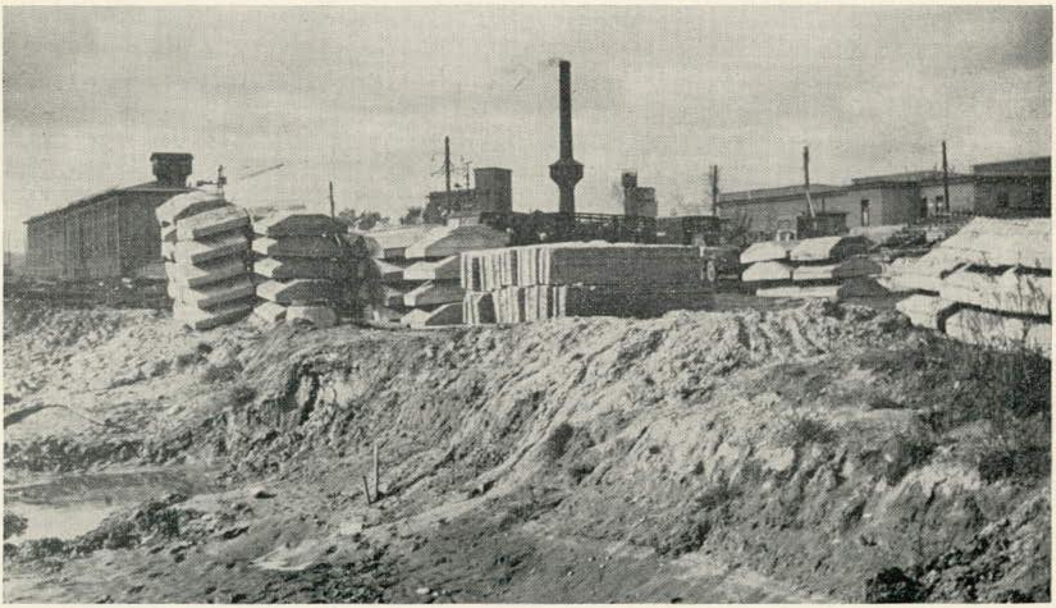


FIGURE 24
Precast footings for apartment buildings, Leningrad.

ment building sites. It was also stated that buildings are sometimes divided into two or three sections to minimize the effects of differential settlement.

At the Leningrad site visited by the delegation several feet of sand were being spread on the bottom of the excavation before the precast strip footings were placed. Precast blocks were set on top of the strip footing to form the foundation walls.

The use of short piles for the foundations of apartment buildings was discussed at the Moscow Institute of Foundation Design and was observed at construction sites in Stalingrad. The piles were about 8 ft long and were spaced at 80 centimeters. A precast reinforced capping beam about 20 in. wide and 12 in. high was placed above the piles.

Another feature of the apartment buildings seen at Stalingrad was the use of silicate concrete in the precast wall slabs. These were made of lime and sand heated to a temperature of 140° C under eight atmospheres of pressure. This material gave a more pleasing surface appearance than the regular concrete slabs.

Efforts are also being made to reduce the amount of settlement of structures by keeping the weights of the structures as low as possible. The use of light weight aggregate aids in accomplishing this purpose.

The delegation visited a plant producing Karamzite, an expanded clay, at the city of Volgski. The clay is heated in rotary kilns to 1000° C. The final product has a specific gravity of 0.4 to 0.45 and concrete produced with it has a unit weight of about 94 lbs per cubic foot. It was stated that the use of this concrete makes the total weight of a building about 25 per cent less than that of similar structures constructed of concrete with regular aggregate.

DEPARTURE

The American delegation's 21-day visit to the USSR came to a conclusion on October 5 when at 9 a.m. the visitors took off from the Moscow airport for the return flight to the United States.

At a formal dinner in Moscow the evening before the visitors from the USA again expressed their thanks and appreciation to the Soviet Host Committee and the associated engineers, soil scientists and interpreters for the many useful presentations made available to the American engineers and educators, and the warm hospitality that marked the exchange mission.

It was indicated that mutual exchanges of Soviet and American specialists could well result in good, reliable and desirable relationships between the engineers and scientists of the two great countries.

VI — Papers Presented by the Soviet Delegation in the USA

A delegation of six Soviet soil scientists and engineers visited the United States, May 31-June 21, 1959, in an initial exchange of experiences in the field of soil mechanics research and its application to foundation engineering. The delegates visited laboratories and observed experiments at several universities and were shown field construction jobs and sites on both East and West Coasts and elsewhere. Their technical papers were presented at five seminars.

The visit was sponsored jointly by the American Society of Civil Engineers, the U. S. National Committee of the International Society of Soil Mechanics and Foundation Engineering, and the Highway Research Board of the National Academy of Sciences-National Research Council.

The Soviet engineers had luncheon in Moscow on May 31 and sat down for supper in New York City the same evening. The time-span limit has been reduced, Ivan M. Litvinov, chairman of the visiting delegation, remarked until the "expansion of communications between the Soviet Union and the United States have become limitless for the development of friendly business and scientific relations by our scientists and specialists . . . and will contribute to a quiet, peaceful life on our, as yet, insufficiently firm earth globe."

The USSR visitors held their first scheduled seminar at Princeton University. This was followed by four other seminars and visits to Rutgers, Columbia and Harvard Universities, the Massachusetts Institute of Technology, Purdue University, the University of Illinois and the University of California at Berkeley. The concluding seminar was held at the National Academy of Sciences in Washington, D. C.

Introductory Remarks
First Soil Mechanics Seminar for the Visiting Engineers from the USSR

FRED BURGGRAF, *Director,*
Highway Research Board,
National Academy of Sciences-National Research Council
(Princeton University, June 3, 1959)

Civilized man has, no doubt, always been cognizant of some of the properties of soils, since he first recognized his footprints in the sand. He built his home of sun-dried brick made from mud. As a progressive-minded farmer, from earliest times to today, he has been keenly aware of the effect of moisture content in the manipulation of soil and has used that knowledge as a guide to commencement of tillage and even to the herding of his animals following wet weather.

As his experience increased and his knowledge accumulated he built earth dams and founded bridges and larger buildings on soil. Slowly he became aware of settlements and of conditions of overstress. Thus, it is always difficult to place one's finger on some specific point in man's progress and say, here was the beginning of soil science or, here the engineer began to use science for the development of what we now know as soil mechanics.

Neither can we single out one nation nor one man as the forefather of soil mechanics, for here as in other scientific endeavors, it is significant that new-found knowledge often springs forth in widely separated places simply as a product of the time and due to different approaches.

There have been some outstanding individuals in the developments in soil mechanics, men who have given shape and dimensions to new ideas. For example, men who developed the pedological system of soil classification, the theory of consolidation, the moisture-density relationship in soil compaction, the present concept of the phenomenon of frost heaving and the currently used concept of swelling pressure, to name only a few. The problems in the field have hardly been more than surface explored.

There are among you who meet here today and who will meet at other points, some of those responsible for new findings which give even better definition to dimensions than those previously used. Those of you who are studying soil freezing and permafrost, the response of soil to vibration and to repeated rapidly applied loadings or impact, exemplify my point. Without doubt, you can name a host of others of equal significance.

Wherever we may now be in our progress of development of this young science of soils mechanics and where our future lies, depends in large measure on individuals now in this room, for you are recognized leaders in the field. That development could take a slow pace if each of you elected to work as an individual and to jealously guard your findings, or that development could be rapid and its benefits be made available to all men quickly if you pool your knowledge through cooperative visits such as this one. The American Society of Civil Engineers and the Highway Research Board of the National Academy of Sciences are organizations whose very existence depends upon cooperation in the solution of engineering problems.

It is my great privilege as director of the Highway Research Board to welcome our distinguished Soviet technical visitors to this the first of five seminars to be held in this country during their three-week visit. We have looked forward to this occasion as one of signal importance and promise. By exchanging ideas and pooling experiences derived from a variety of studies in the two countries we can do much in furthering our knowledge of soils in the field of engineering and in making possible similar future meetings of mutual benefit.

In conclusion, I would like to express my sincere gratitude to each of you who have come to contribute your ideas and experiences, for from a better knowledge of soil mechanics will come better solutions to our soil engineering problems.

Problems of Frozen Soil Mechanics in Engineering Practice

NIKOLAI A. TSYTOVICH, *Chairman,*
National Association of Soil Mechanics and Foundation Engineering, USSR

SUMMARY OF THE REPORT

More than a quarter of the world's land and about 47 per cent of the USSR territory are covered by permanently frozen soils *(Fig. 1)*.

Construction on permanently frozen soils is associated with many specific features, which, if not taken into consideration, cause unadmissible deformations quickly destroying structures *(Fig. 2)*.

The deformations of structures built on permanently frozen soils are due to their settling at unproportional thawing and the heaving of soils and foundations at freezing *(Figs. 3, 4, 5, 6, 7)*.

These deformations are the results of peculiar properties of frozen soil and are caused by sharp alterations of their volume and structure both at freezing and thawing *(Fig. 8)*.

The proper solution of the problems of construction on permafrost is possible on the basis of a new science—mechanics of frozen soils—the application of which allows safe and economical design of the constructions.

The main contemporary problems are:

1. Consideration of the parameters and the determination of stress-strained state of frozen soils.

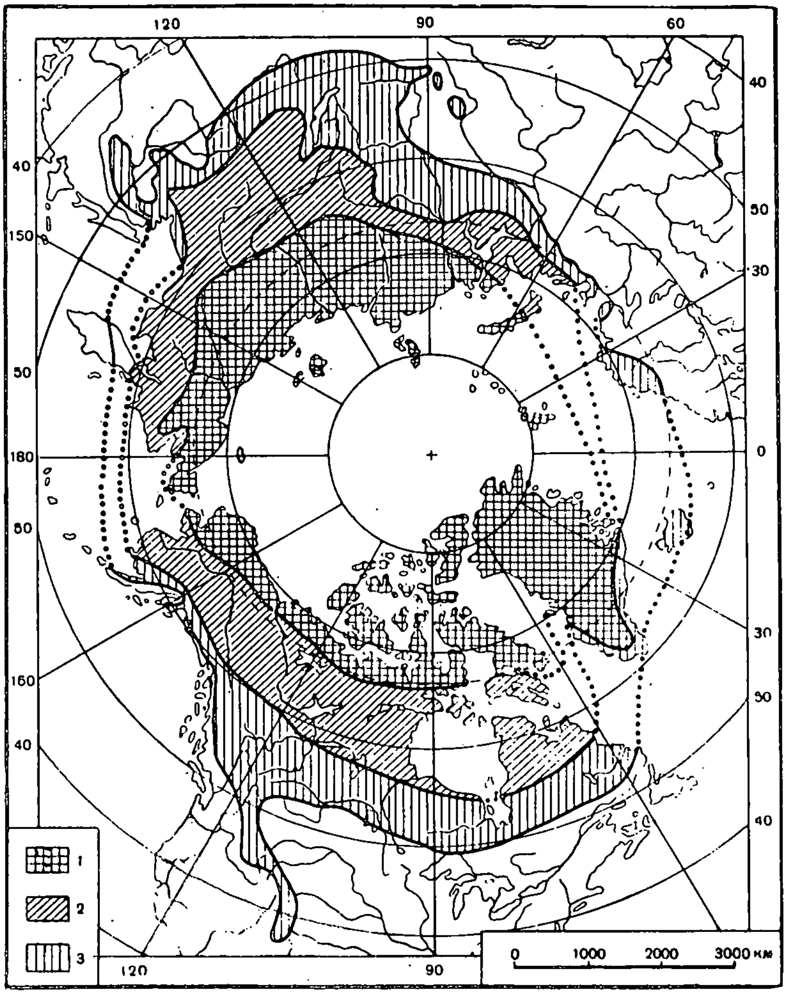


FIGURE 1

Distribution of frozen soils in the northern hemisphere (after Black): 1. zone of continuous permafrost; 2. zone of discontinuous permafrost; 3. zone of sporadic permafrost.

2. Establishment of general laws of frozen soils mechanics by means of thorough study of their nature.

3. The methods of solution of frozen soil mechanics, and the engineering problems for construction practice.

4. The investigation of the practical applicability limits of separate theories and the determination of the corresponding correction coefficient.

The most important parameters of frozen soils are as follows:

1. Shearing strength, as the initial value for limit load and deformation modulus of frozen soils determination, when they are considered as foundations *(Fig. 9)*.

2. Continuous strengths and deformation moduli of frozen soils, as construction materials *(Fig. 10)*.

3. Heaving forces, congelation strength, "solidity coefficients" of frozen soils as a medium for construction *(Fig. 11)*.

It is presently necessary to pass from local characteristics of frozen soils to the characteristics of frozen soil massifs and to use new methods: crystallooptical, radioactive rays, ultrasound, electrotensometric and other methods.

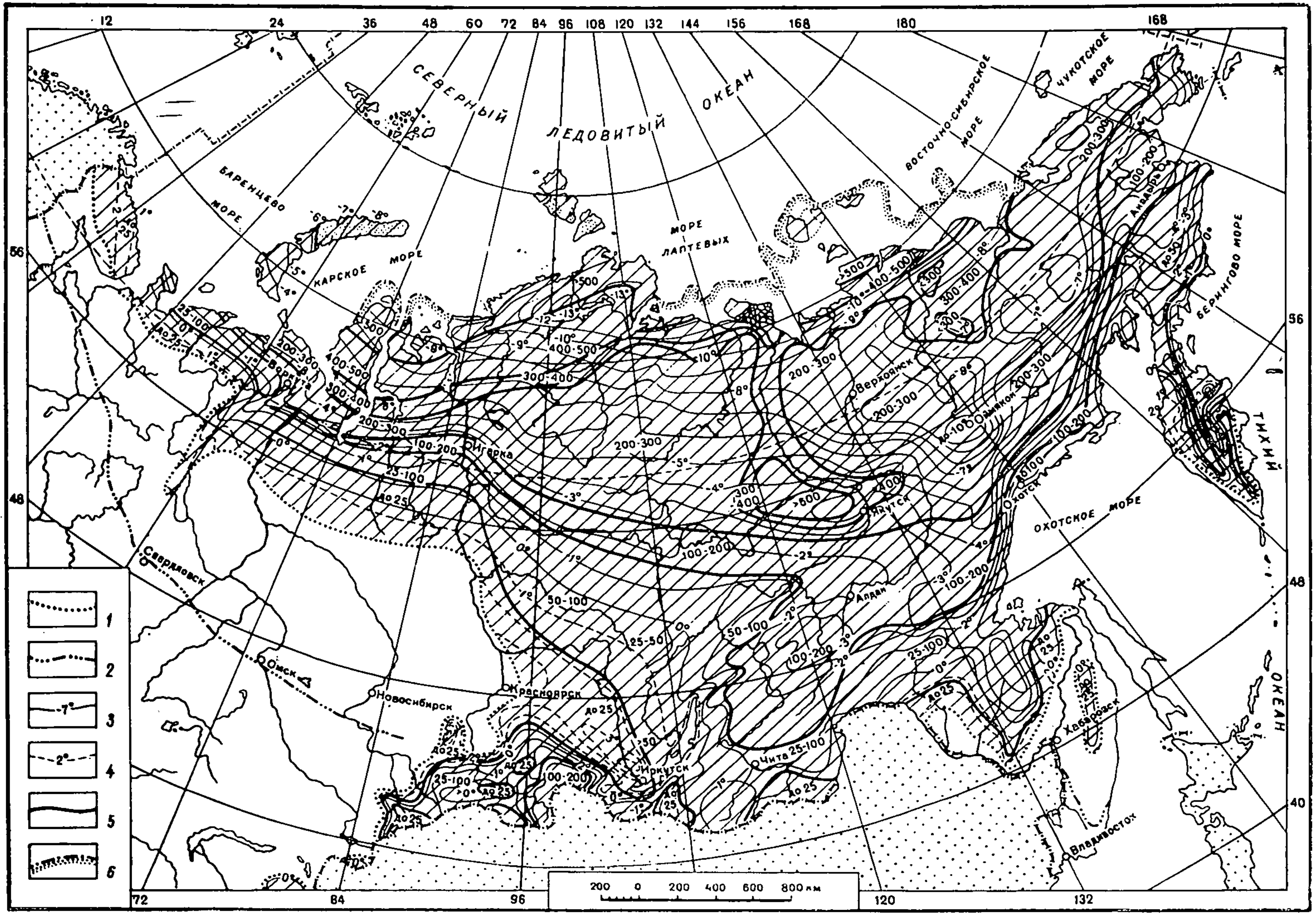


FIGURE 2

Permafrost map of the USSR (after Baranov): 1. southern boundary of permafrost occurrence; 2. boundary of the "pereletki" zone; 3. temperature isolines at the depth of zero amplitudes; 4. temperature isolines in soil at the depth of 1 to 2 meters; 5. thickness of permafrost (in meters); 6. permafrost boundary under sea bottom.

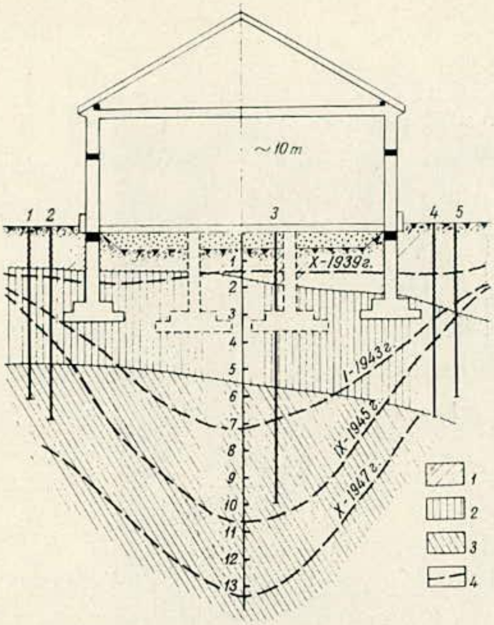


FIGURE 3

An example of unproportional soil thawing as the cause of foundation deformation: 1. mantle clayey soils; 2. moraine clayey soils; 3. gravel clayey soils (lower moraine); 4. boundary of thawing ($S = 20$ cm; $\Delta S = 8$ cm; occurrence of fissures).

Experimental investigations allow establishment of the following cardinal points of frozen soil mechanics:

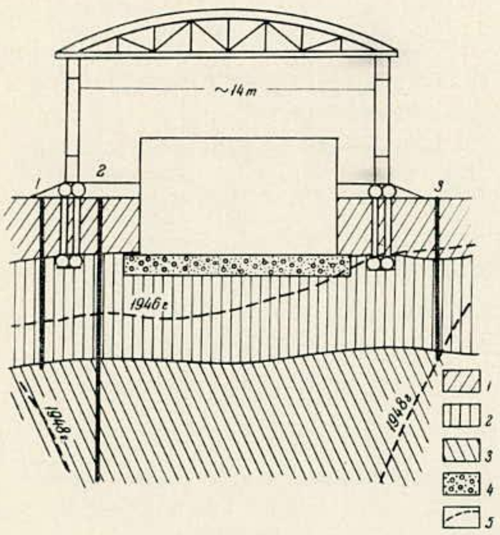


FIGURE 4

Thawing of soils under the boiler building: 1. mantle clayey soils; 2. moraine clayey soils; 3. moraine clayey soils of lower moraine; 4. gravel; 5. thawing boundary.

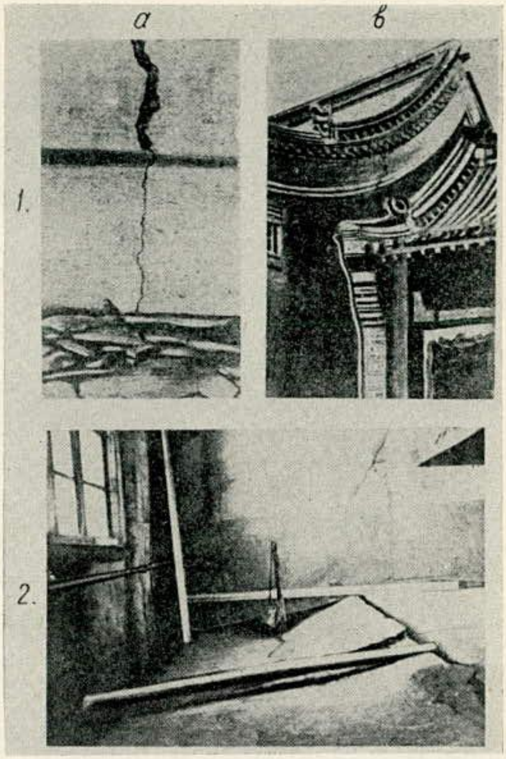


FIGURE 5

Photos of deformed building erected on permafrost soils: 1. fissures in randbeams (a) and in building walls (b) due to the heaving of soils; 2. floor deformation at fish packing plant caused by settling of frozen soils on thawing.

1. The principle of dynamic balance of water and ice in frozen soils *(Fig. 12)*.

2. The conditions of water migration in freezing and frozen soils *(Fig. 13)*.

3. Dependence of frozen soil strengths on their composition, temperature and structure *(Fig. 14)*.

4. Decrease of frozen soil strength to external forces with time due to the relaxation of stresses *(Fig. 15)*.

5. The conditions of frozen soil densification and the beginning of plastic flow *(Fig. 16)*.

6. The dependence of frozen soils settlements at thawing on the value of external pressure *(Fig. 17)*.

7. The breaking of structural bonds in freezing-thawing cycles *(Fig. 18)*.

The chief practical engineering problems of frozen soil mechanics are:

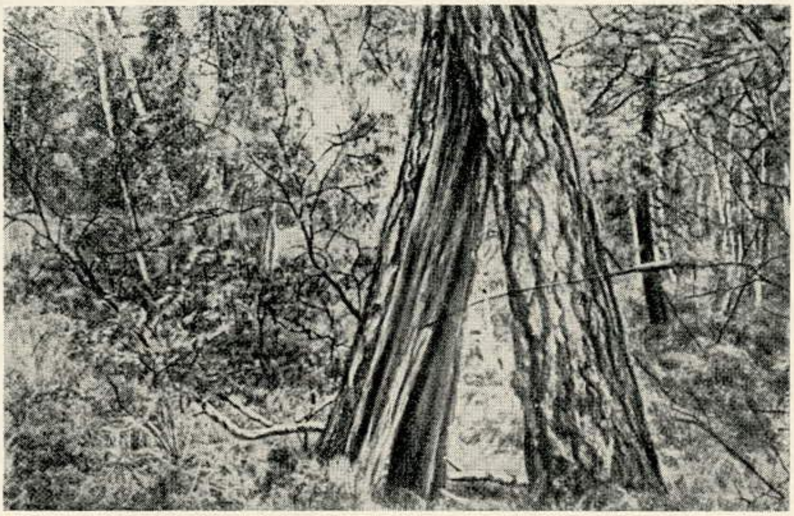
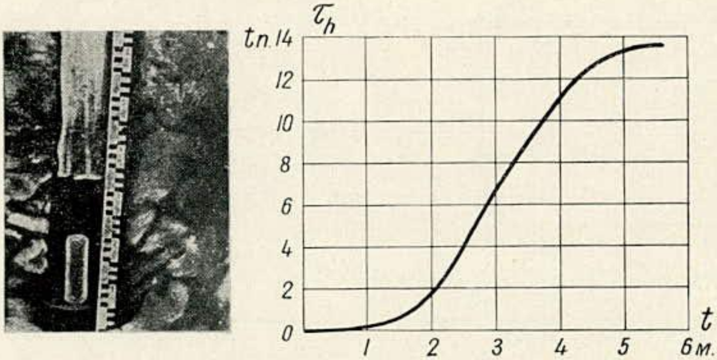


FIGURE 6
An illustration of frost heaving forces in soils (splitting of tree trunk).



1. 2.

FIGURE 7
Open ground works with upheaved post: 1. upheaved post; 2. change of frost heaving forces with time.

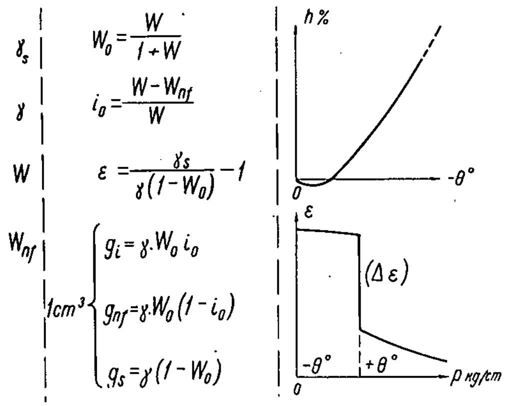


FIGURE 8

Physical properties of frozen soils. Determined by measurements: γ_s—specific weight of mineral particles; γ—volume weight of natural structure; W—natural water content; W_{nf}—quantity of unfrozen water. Calculated: W_0—total water content (relative to wet soil weight); t_0—relative ice content, ϵ—porosity coefficient, g_i—ice weight; g_{nf}—unfrozen water weight, g_s—mineral particles weight; at the right side—heaving curves at freezing and settling at thawing.

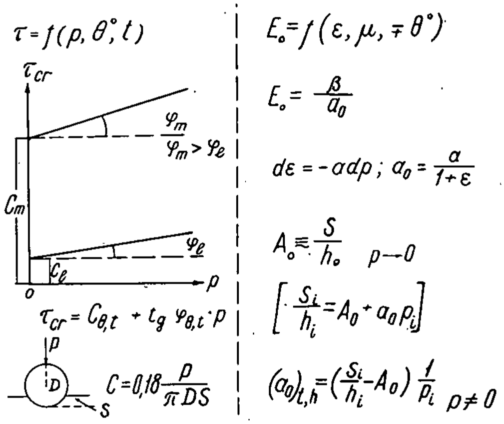


FIGURE 9

Characteristics of frozen and thawing soils as construction bases. τ_{cr}—critical (limiting) shearing strength; C_m, ϕ_m—instantaneous cohesion and internal friction angle; C_e and ϕ_e—corresponding continuous values; E_0—general deformation modulus, A_0—reduced thawing coefficient; a_0—reduced coefficient of densification at thawing.

Fr.gr. σ_e кg/cm²

fr. > 0,005mm	$-\theta°$		
	-0,4°	-1,2°	-4,0°
< 3%	6	10	14
3-10% (W < 35%)	3,5	7	10
10-30% (W < 45%)	3,0	5	8
w.i. (W > 45%)	2,5	4	6

Ice

$R_\alpha = 5 - 150$ кg/cm²

$R_{m\alpha} \approx 30$ кg/cm²

$\eta \approx 2{,}5 \cdot 10^{14}$ gr.sec/cm² $= 7 \cdot 10^{14}$ кg.h./cm²

$R_z \approx \frac{1}{2} \cdots \frac{1}{3} R_\alpha$

$$\varepsilon = \varepsilon_c + \varepsilon_e + \varepsilon_p$$

FIGURE 10

Frozen soil and ice strength, as construction materials. σ_e—continuous strength (strengths for designing); R_d—destruction pressure stress; R_z—the same for tension; ϵ—relative deformation; of densification (ϵ_c), elastic (ϵ_e), and plastic (ϵ_p).

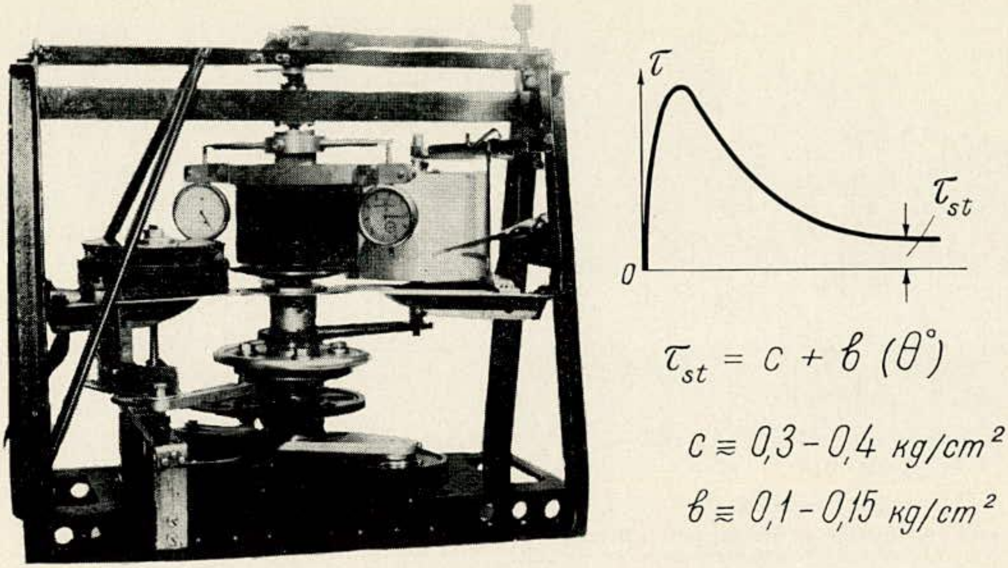


FIGURE 11

Characteristics of frozen soil as a construction medium. (Left) B. Dalmatov's apparatus for determination of heaving forces. (Right) determination of steady congelation force (τ); and continuous congelation strength (τ_{st}).

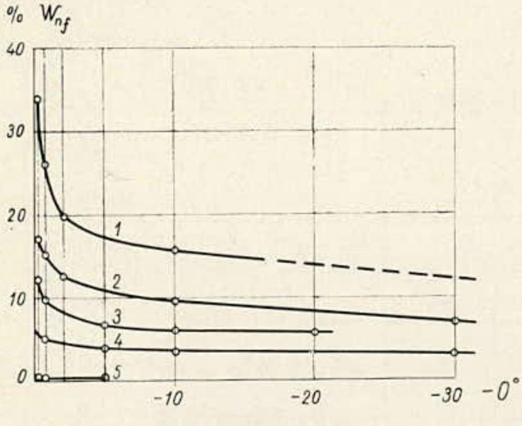


FIGURE 12

Curves of equilibrium content of unfrozen water (W_{nf}) in frozen soils. 1. Clay; 2. cover clay; 3. loam; 4. clayey sand; 5. sand.

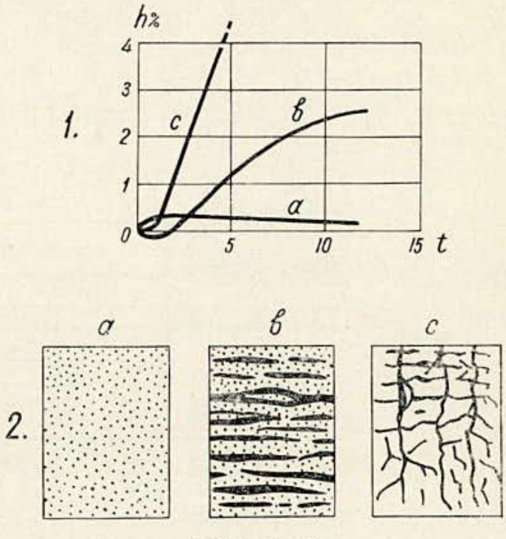


FIGURE 13

Processes caused by the migration of water in freezing soils (frost heaving, formation of structure). 1. Frost heaving of soils (a) sand, (b) clay, (c) silty loam with supply of water; 2. types of frozen soil structures (a) massive; (b) laminar; (c) cellular.

1. Prognosis of temperature and moisture field in frozen soils and the changes under the influence of constructions *(Figs. 19, 20)*.

2. Determination of limit loads for frozen soils *(Fig. 21)*.

3. Determinations of frozen soil deformations at different stages of strain (at densification, plastic and progressing flow as shown in Figure 16).

4. Prognosis of construction settlements on thawing soils as shown in Figure 17.

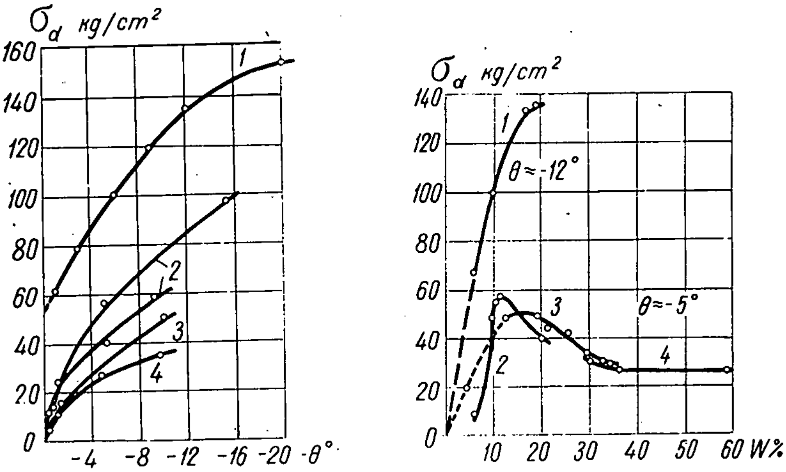


FIGURE 14

Dependence of frozen soil strengths on composition, temperature and structure. (Left) dependence of limiting strength (σ_d) of frozen soils on pressure: 1. sand, 2. clayey sand differently wetted; 3. clay; 4. silty clay. (Right) dependence of limiting strength (σ_d) on water content.

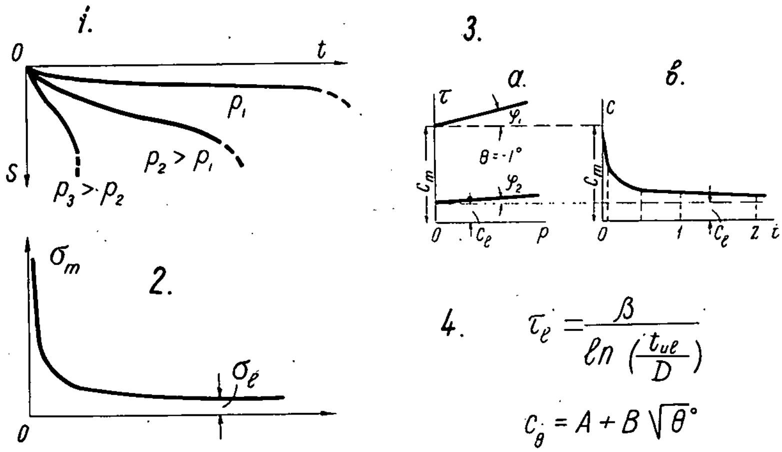


FIGURE 15

Frozen soil strength decrease with time (relaxation of strength). 1. Transition of plastic flow into progressive flow; 2. decrease of normal strength; 3. decrease of shearing strength; 4. dependence of shearing strength (τ) on time, and cohesion (C_θ) on the value of negative temperature.

5. Consideration of the alterations in mechanical properties of soils when the soils freeze and thaw as shown in Figure 18.

Let us consider briefly some general solutions of engineering problems concerning the mechanics of frozen soils including the calculations of foundations with preservation of permafrost according to the constructive method *(Fig. 22)*, taking into account the thawing settlements *(Fig. 23* and *Figs. 31, 32)*, and by the method of preconstruction thawing *(Figs. 24, 25)*.

The limits of applicability of suggested methods and worked-out solutions can be established only by means of special observations of the stress-strained state and its

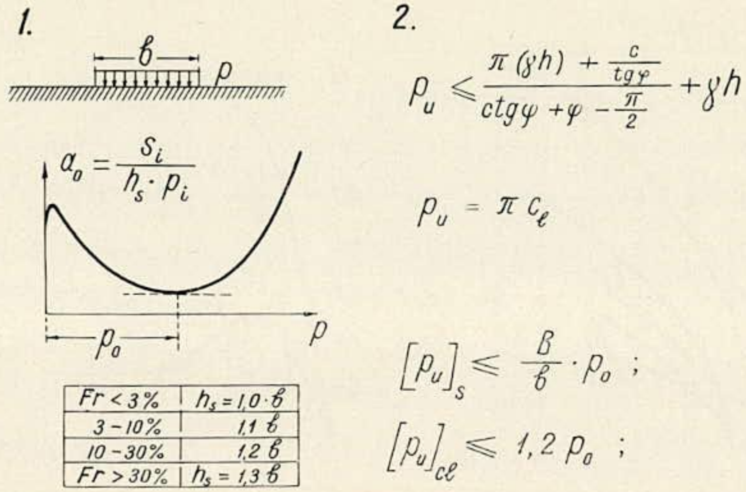


FIGURE 16

Conditions of soil densification and of plastic flow start. 1. Determination of densification stage limit (p_0 kg/cm²) by testing load results; 2. theoretical calculation of densification stage limit ϕ—internal friction angle, c—cohesion, B—foundation width, b—testing area width, a_0—reduced densification coefficient; c_e—continuous cohesion for cohesive soils, $p_{u,s}$—for sands, $p_{u,cl}$—for clays.

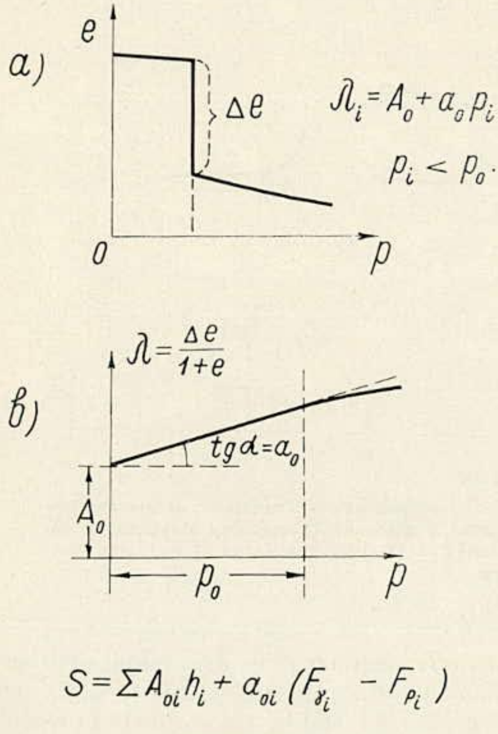


FIGURE 17

Dependence of relative deformation (λ_i) of thawing soils and of total formation settling (S_t) on external pressure (p_i) and own weight (γ) of thawed soil. (F_{γ_i}—area of diagram of densifying pressure caused by own weight of thawed soil; F_{pi}—the same for external load.

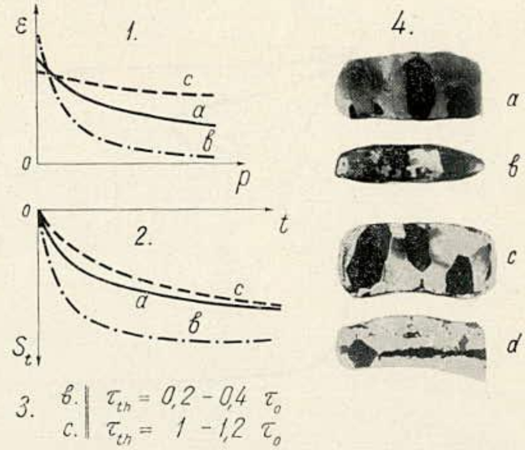
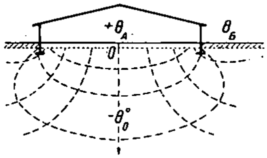


FIGURE 18

Changes in structural bonds in soils during freezing-thawing cycle. 1. Compressibility change; 2. densification speed change; 3. shearing strength change during freezing-thawing cycle: (a)—before freezing, (b)—for laminar and cellular structure, (c)—for massive structure at freezing; 4. ice structure: (a, c)—before deformation, (b, d)—after compression and after shearing.

changes in the course of time in experimental constructions with the determination of mechanical parameters of frozen soils and of construction materials.



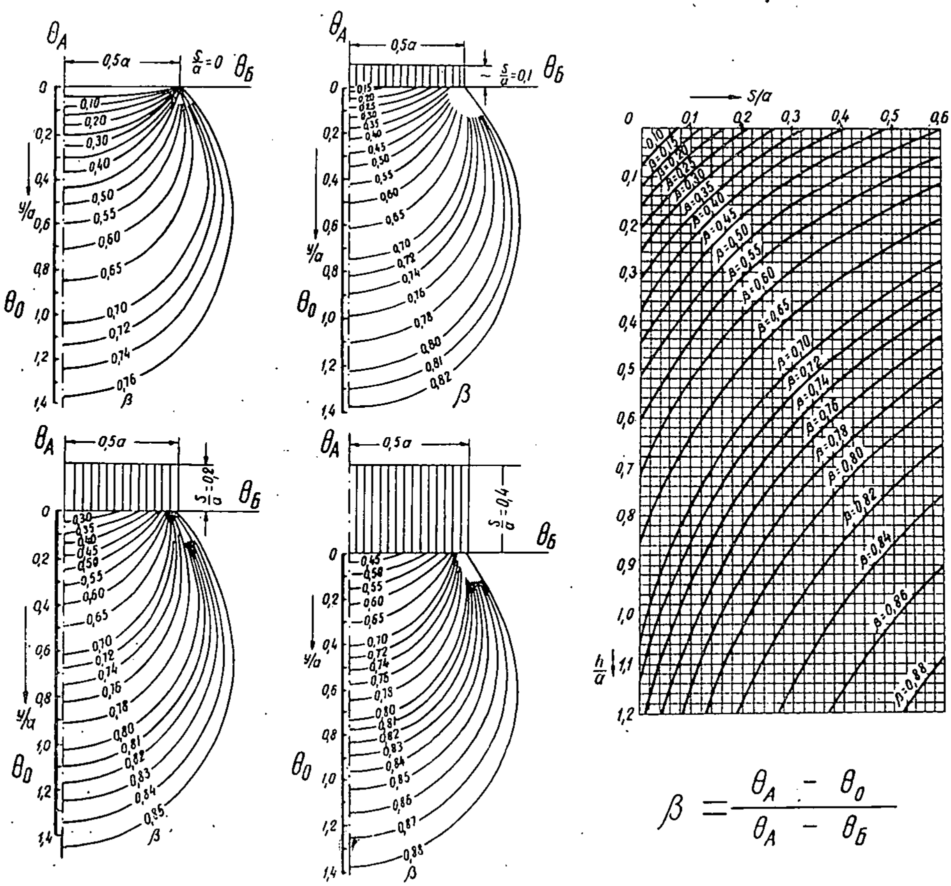
$$h_{max} = a\sqrt{t} - \beta t$$

$$a = \sqrt{\frac{2\lambda(\Delta\theta)}{\xi \cdot W_0 \gamma \cdot i_0 + \frac{c\gamma(\Delta\theta)}{2}}} \; ; \; \beta = \frac{q}{\xi \cdot W_0 \cdot \gamma \cdot i_0}$$

$$h_0 \approx h_{max}$$

FIGURE 19

Thawing depth determination. h_{max}—thawing depth in the case of linear problem; λ—coefficient of heat conductivity; C—heat capacity; ξ—ice melting heat; q—thermal flow from the Earth interior (0.5 to 6 kcal/m²/hr.



$$\beta = \frac{\theta_A - \theta_0}{\theta_A - \theta_б}$$

FIGURE 20

Calculation of limit thawing "basin" (after Golovko) S/a—equivalent thermal isolation thickness, b—width.

	1.	2.			
	$c=f(t,W,\theta^\circ);\varphi=0$	$c=0;\varphi=f(t,W,\theta^\circ);p_u=D\cdot\gamma b+\gamma h$			
		h/b	$\varphi=26^\circ$	30°	36°
$\frac{l}{b}>10$	$p_u=5{,}14c+\gamma h$	0	6,8	D = 10,8	26,2
		1	21,3	34,8	79,5
		2	36,3	58,9	138,0
$\frac{l}{b}=1$	$p_u=5{,}7c+\gamma h$	0	—	17,3	~56
		0,5	—	33,7	~106
		2	—	122,0	~335
$\frac{l}{b}>10$	$p_u=c\cdot Ctg\,\varphi(\zeta-1)+\gamma h\cdot\zeta;\ \zeta=e^{\pi tg\varphi}tg^2(45^\circ+\frac{\varphi}{2})$ $\varphi\neq0;\ c\neq0$				

FIGURE 21

Formulas for determination of limit load on frozen and thawing soils. 1. For frozen cohesive and solid clayey soils; 2. for thawing soils ($\phi_{t,h} < \phi$) and for dense loose soils.

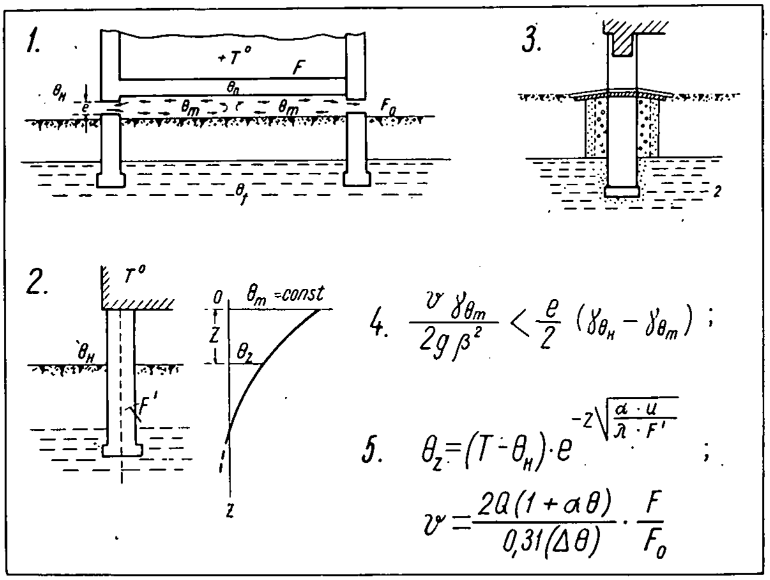
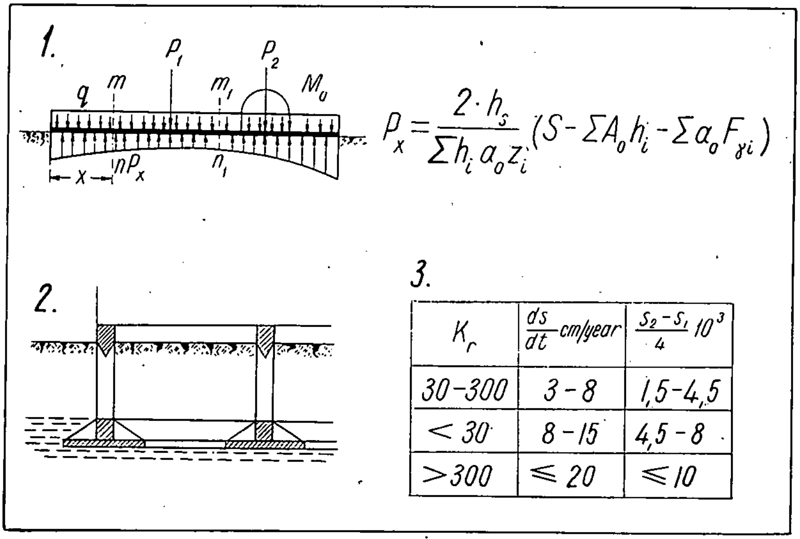


FIGURE 22

Calculation of foundations erected with preservation of permafrost in base. 1. Scheme of underfloor space ventilated in winter; 2. temperature change with depth of pillar foundation; 3. constructive scheme of foundation and counter-heaving filling; 4. formula for calculation of height (e) of ventilated underfloor space; 5. formula for determination of temperature along foundation axis.



$$P_x = \frac{2 \cdot h_s}{\sum h_i a_0 z_i} (S - \sum A_0 h_i - \sum a_0 F_{\gamma i})$$

K_r	$\frac{ds}{dt}$ cm/year	$\frac{S_2 - S_1}{4} 10^3$
30–300	3–8	1,5–4,5
< 30	8–15	4,5–8
>300	≤ 20	≤ 10

FIGURE 23

Schemes of calculation and construction of foundations erected with the thawing of frozen base soils. 1. Calculation scheme; 2. construction scheme for rigid foundations; 3. value of limit deformations at base thawing. K_r—rigidity coefficient.

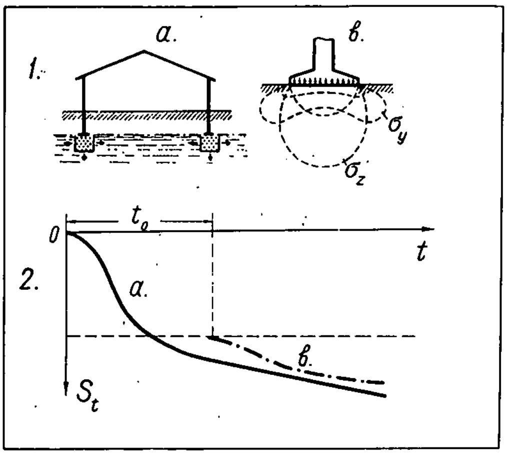


FIGURE 24

Scheme of foundation work at preliminary thawing: 1. (a) thawing zones scheme, (b) pressure isolines under foundation; 2. settling value: (a) in the process of thawing at construction exploitation and (b) at preliminary thawing; t_0 time of preliminary thawing.

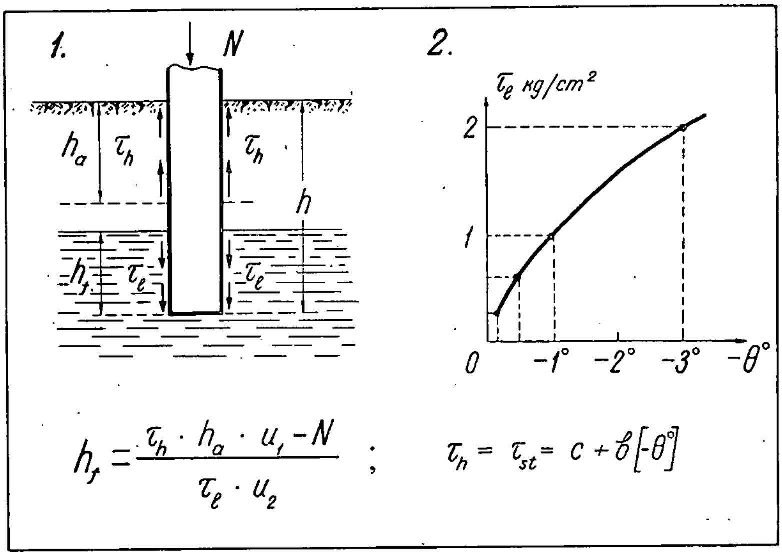


FIGURE 25

Scheme of foundation calculation for frost heaving. 1. Forces involved; 2. dependence of continuous congelation strength (τ_e) on the value of negative temperature; 3. formulas for necessary foundation sinking into frozen soil; h_a—active depth, about $\frac{2h}{3}$ in the case of frozen soils; U_1, U_2—formulation perimeter in active layer and in permafrost; τ_h—specific upheaving force.

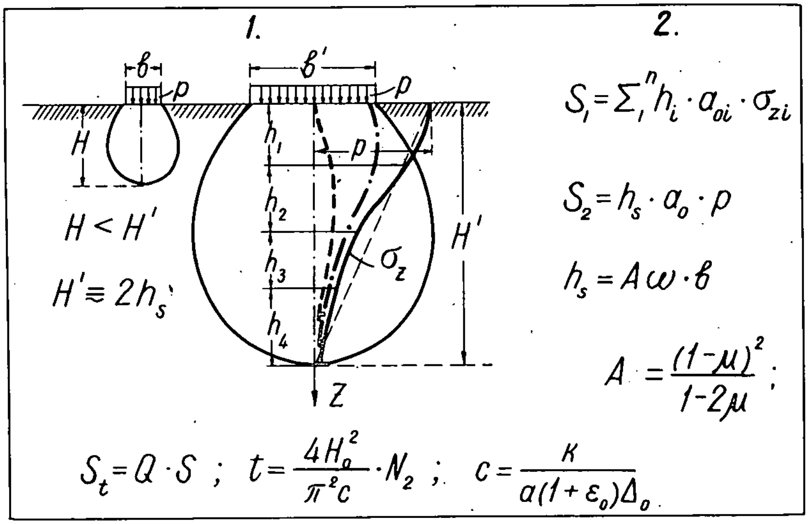
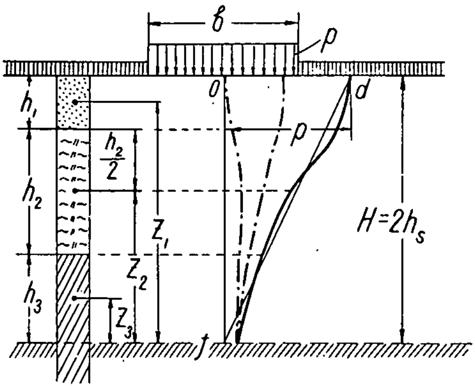


FIGURE 26

Example of foundation setting calculation on homogeneous soils: 1. dependence of densification zone value on dimensions of loading area and the concept of equivalent soil layer (h_s); 2. formulas for foundation setting calculation in homogeneous soils; S_1—by the method of elementary summing (without accounting for lateral expansion of soil); S_2—by the equivalent layer method (worked out by the author) accounting for lateral expansion of soil (μ), and size and shape of foundation, and its rigidity (coefficients, w, b).



$$S = h_s \cdot a_{om} \cdot p$$

$$a_{om} = \frac{\Sigma h_i \, a_{oi} \, z_i}{2h_s^2}$$

$$K_m = \frac{H}{\Sigma \frac{h_i}{K_c}}$$

$$\mathcal{E}_m = \frac{H - H_0}{H_0}$$

FIGURE 27

Equivalent diagram of densifying pressures (odf) and formulas for calculation of foundation settlement on laminar layer of soils by the equivalent soil layer (h_s) method.

$a = \frac{l}{b}$	Gravel		Sands				Plastic Loams		Plastic Clays	
	Hard Clay and Loams				Sandy Loams					
	$\mu = 0.10$		$\mu = 0.20$		$\mu = 0.25$		$\mu = 0.30$		$\mu = 0.35$	
	Aw_c	Aw_m	Aw_c	Aw_m	Aw_c	Aw_m	Aw_c	Aw_m	Aw_c	Aw_m
1	0.568	0.96	0.598	1.01	0.631	1.07	0.687	1.17	0.790	1.34
1.5	0.687	1.16	0.724	1.23	0.764	1.30	0.832	1.40	0.956	1.62
2	0.775	1.31	0.817	1.39	0.862	1.47	0.938	1.60	1.079	1.83
3	0.903	1.55	0.951	1.63	1.003	1.73	1.092	1.89	1.256	2.15
4	0.994	1.72	1.047	1.81	1.105	1.92	1.203	2.09	1.383	2.39
5	1.065	1.85	1.122	1.95	1.184	2.07	1.289	2.25	1.482	2.57
6	1.124	1.98	1.184	2.09	1.249	2.21	1.360	2.41	1.568	2.76
7	1.173	2.06	1.236	2.18	1.304	2.31	1.420	2.51	1.632	2.87
8	1.216	2.14	1.281	2.26	1.316	2.40	1.472	2.61	1.692	2.98
9	1.254	2.21	1.321	2.34	1.393	2.47	1.517	2.69	1.744	3.08
10 and more	1.288	2.27	1.357	2.40	1.431	2.54	1.558	2.77	1.792	3.17

FIGURE 28

Table of equivalent layer coefficient (Aw) values for calculation of foundation settlement on compressible soils: Aw_c—for corner points of flexible foundations with rectangular bases; Aw_m—for average settlement of rigid foundations.

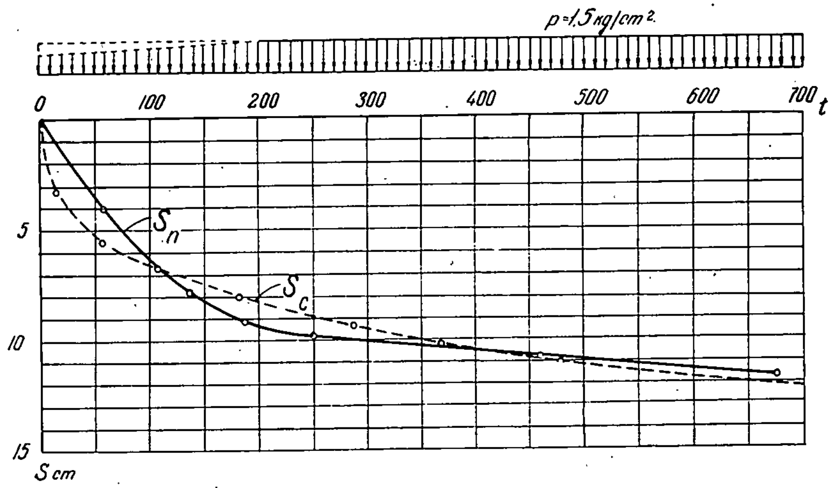


FIGURE 29

Comparison of settlement (S_c) calculated by equivalent layer method with measured settlement (S_n) of school building erected on complicated (seven layers) soil base.

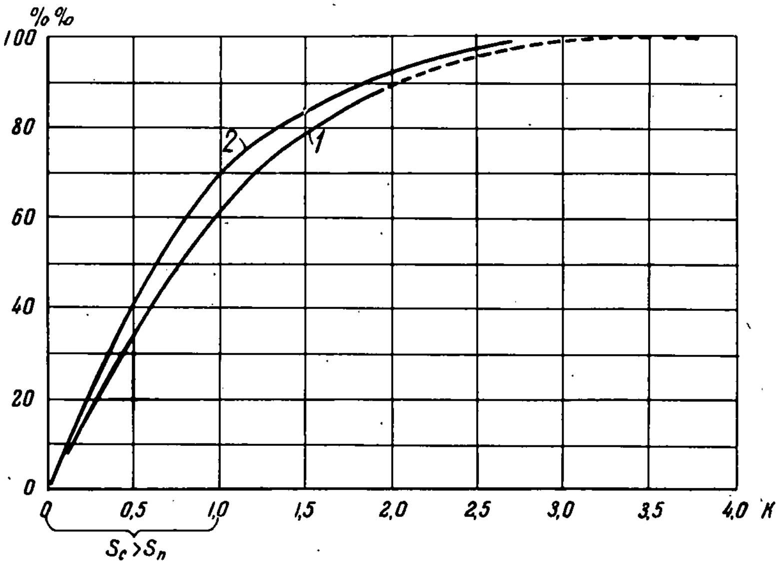
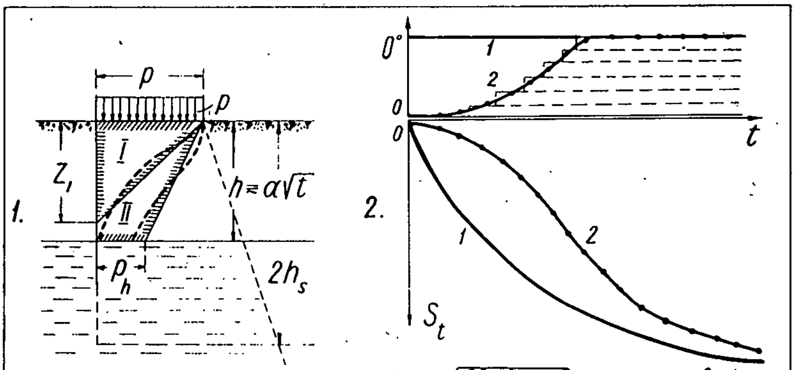


FIGURE 30

Curves of statistical distribution of certainty coefficient (K) for different methods of foundation calculation: 1. as calculated by the method of elementary summation; 2. as calculated by the equivalent soil layer method (when design settlement in 80 per cent of the cases is not smaller than actual, coefficient of certainty is from 1.3 to 1.5).



$$S_{tI} = A_0 h + \frac{a_0 z_1}{2} p \qquad z_1 = \frac{B\gamma}{2} + \sqrt{\frac{B^2\gamma^2}{4} + Bp}\ ; \ B = \frac{6\kappa t}{\Delta_0(3A_0 + a_0 p)}.$$

$$S_{tII} = A_0 h + \frac{p + p_h}{2} a_0 h \qquad p_h = p(1 - X)\ ; \ \kappa < \kappa_\nabla \text{ -- } C\ I\ ; \ \kappa > \kappa_\nabla \text{ -- } C\ II$$

$$\frac{\Delta_0 a_0 h^2 - 6\kappa t}{2\Delta_0 a_0 h^2} = \frac{6\kappa t\gamma h - 3A_0\Delta_0 h^2}{2\Delta_0 a_0 h^2 p X} - \ln X\ ; \ \kappa_\nabla = \frac{(3A_0 + a_0 p)\Delta_0 h^2}{6t(p + \gamma h)}$$

FIGURE 31

Calculation scheme for causes of change with time of foundation settlement in thawing soils: 1. when thawing speed is greater than that of densification (S_{tI}) and 2. when speed of densification is less than that of thawing (S_{tII}); on the right—curves of foundation settlement with time at a different heat transfer. $\theta = F(t)$ in partial case at $A_0 = 0$ and $\gamma h = 0$ the formulas transform into those obtained previously by K. Terzaghi for unfrozen soils.

Ultimate values of settlements

K_r	$\frac{dh}{dt}$ m/year	S cm	$\frac{dS}{dt}$ cm/year	Slope $i \cdot 10^3$	Deflection $f \cdot 10^3$
1	2	3	4	5	6
30÷300	0,3-0,9	10-20	3-8	1,5-4,5	1-3
<30	0,9-1,8	20-40	8-15	4,5-8	3-6
≥300	to 2	to 50	to 20	to 10	—

$$K_r \cong 1{,}7(1 - \mu_0^2)\left(\frac{A_0}{p} + a_0\right)\cdot\Omega\cdot E_B\frac{H^3}{L^3}$$

FIGURE 32

Ultimate values of settlement (after V. P. Ushkalov). 1. Rigidity index; 2. soil thawing rate under the construction; 3. settlement rate; 4. limiting settlement rate; 5. limit gradient; 6. limit downwarping of foundations (according to USSR standards, values of admitted deformations are about twice smaller).

Equipment for Field Geotechnic Investigation of Soils

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Academy of Construction and Architecture, Ukrainian SSR

Problems of field investigations of structural properties of soils have an important practical significance for various types of construction. At the present time in the USSR and in the USA there are several different methods for the solution of these problems. In the United States and the West European countries various instruments and methods based on penetrometers of relatively small size are used for field investigations.

In the USSR there are two methods of soil investigations utilizing pits and bore holes. The first method utilizes loading by presses and platforms of relatively large size, many exceeding the size of the penetrometers used in the United States and West Europe. In the second method, establishment of carrying capacity of soils is carried out on the basis of definite physical and mechanical properties of soils obtained from undisturbed samples.

My remarks are directed to the second method. In this connection I shall permit myself briefly to acquaint you with some of the instruments developed by me which have been manufactured and used in the USSR and other countries during the past eight years. The techniques of using these instruments are described in a book, "Investigations of Soils Under Field Conditions" and the corresponding officially approved specifications.

In the Southern Building Research Institute YUZHNII there have been developed Litvinov Type-9 field laboratories for accelerated tests of the building properties of soils directly on the site. The laboratories are in serial production now and are widely used both in the USSR and in other countries. They are portable sets of equipment conveniently operated and handled by one man and can replace the complicated and bulky equipment of stationary field laboratories. The equipment has been thoroughly tested both in laboratories and on the site by research and productive institutions and has been used with success.

The equipment of these field laboratories permits the determination of the physical and mechanical properties of soils to be made with a high degree of accuracy by drawing conclusions from experimental data based on parallel field tests of hundreds of soil block samples rather than on testing a limited number of samples as is usually done. The time required for taking undisturbed samples and making accelerated determinations of their basic physical properties has been reduced to one-tenth of that required for other generally employed methods and apparatus. The expenses involved in carrying out research work are thus considerably reduced and the rate at which it is done is greatly increased.

The field laboratories which are contained in two hand cases weighing eight and 12 kgs each and are equipped with a drying cabinet (1.5 kgs weight) and a device for shear tests of soil weighing 6 kgs *(Figs. 1, 2)*. The equipment also includes devices and apparatus of special design that permit determinations of all the building properties of soils required by the codes.

Here are some examples of the use of this equipment:

(a) Selection of undisturbed soil block samples of natural moisture content from holes and pits by using the devices shown *(Figs. 3, 4)*. These block samples are then used to determine the basic physical properties of the soil and tested for filtration, compressive properties, shear, etc.

(b) Determination of various soil characteristics: dry volume-weight, wet volume-weight, moisture content; porosity, density, and permeability under various loadings; plasticity, consistency, granulometricity, angles of the slope of repose of sandy soils *(Fig. 5)*, swelling, relative modulus of compression, structural cohesion, shearing cohesion of plastic clayey and silty soils, etc.

(c) Determination of the compressive properties by the usual or accelerated method in a consolidometer *(Fig. 6)* or

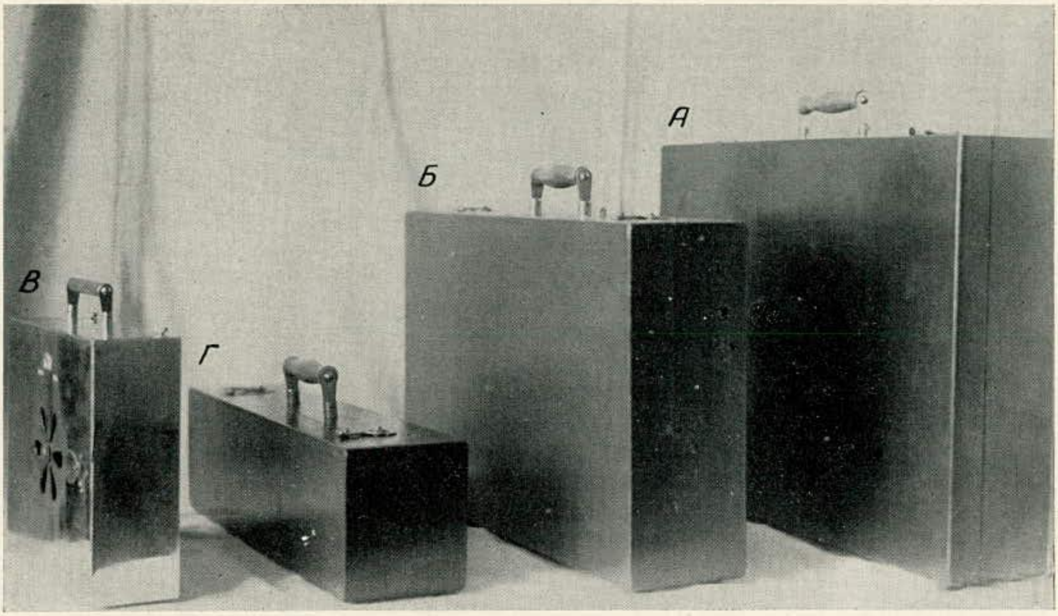

FIGURE 1
General view of the Litvinov Type-9 field laboratory developed in YUZHNII: A. Main set of equipment; B. compression device; C. field drying oven; D. shear test device.

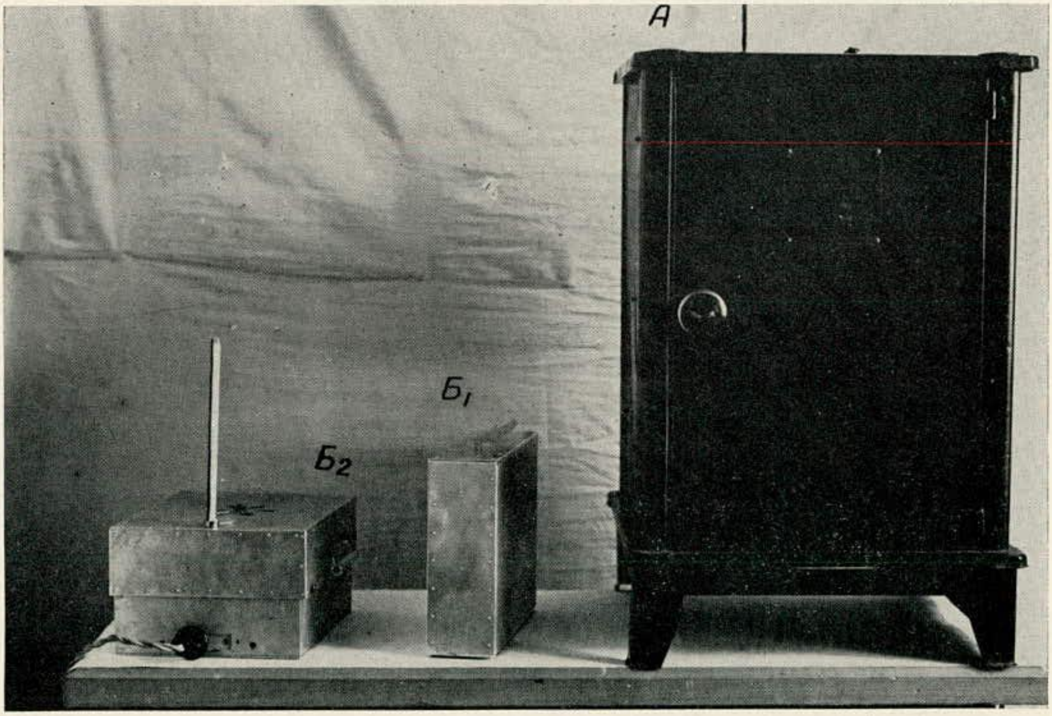

FIGURE 2
Drying oven. A. Standard oven for laboratory work; B-1. oven for Type-9 field laboratory ready for shipment; B-2. oven in working condition.

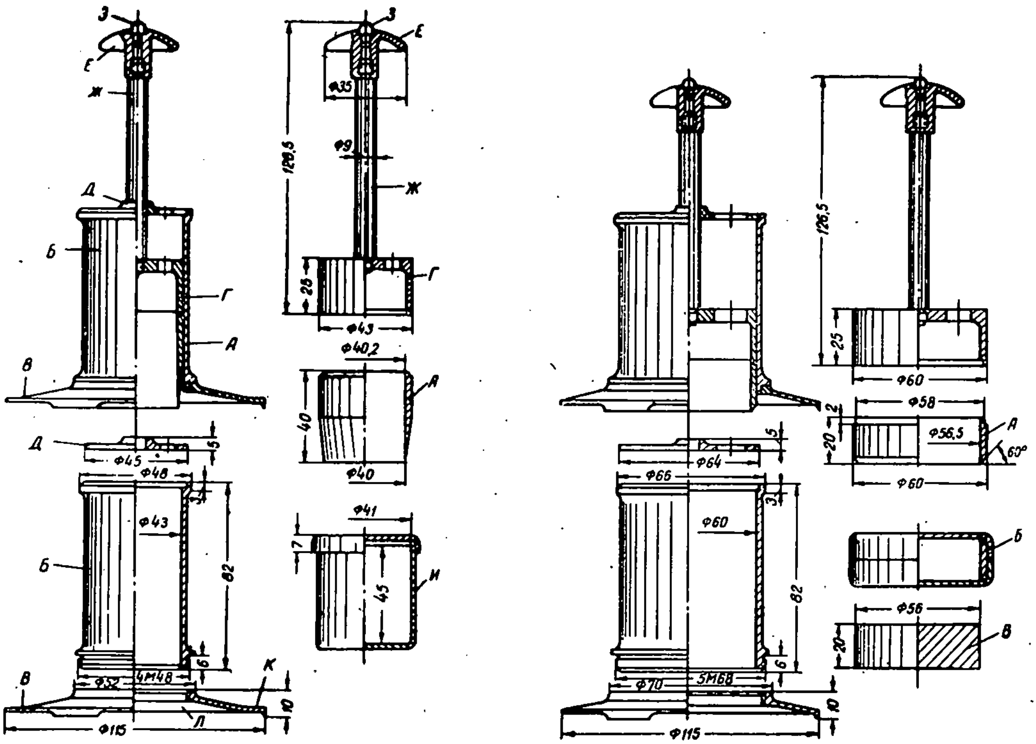

FIGURE 3
Type-9 field laboratory devices used in collection of undisturbed soil samples for determination of the basic physical, compressive, sedimentary, filtration and shear properties.

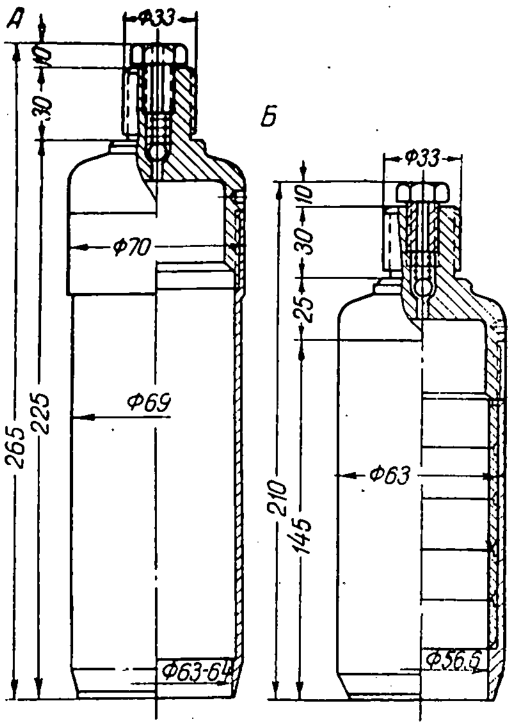

FIGURE 4
Soil sample transporters D-1 and D-2.

another device *(Figs. 7, 8)* by means of which pressure is applied uniformly on all sides of the block sample.

(d) Determination of the structural properties of loess soils, the coefficients of settlement after wetting *(Fig. 6)* and the size of possible settlements after wetting.

(e) Field investigation of the shear properties of soils by means of a special device *(Figs. 9, 10, 11)*.

(f) Determination of the critical pressure on the soil in accordance with the codes and technical requirements.

(g) Calculation of the possible settlement of the footings of various buildings and structures.

This equipment ensures proper control of the quality of different earthwork in the construction of soil dams, embankments, dikes, canals, fills and other structures involving the use of soils *(Fig. 12)*.

The drying of soil samples is carried out in a collapsible oven of special design (see Figure 2) weighing only 1.5 kg and having dimensions very convenient for transporta-

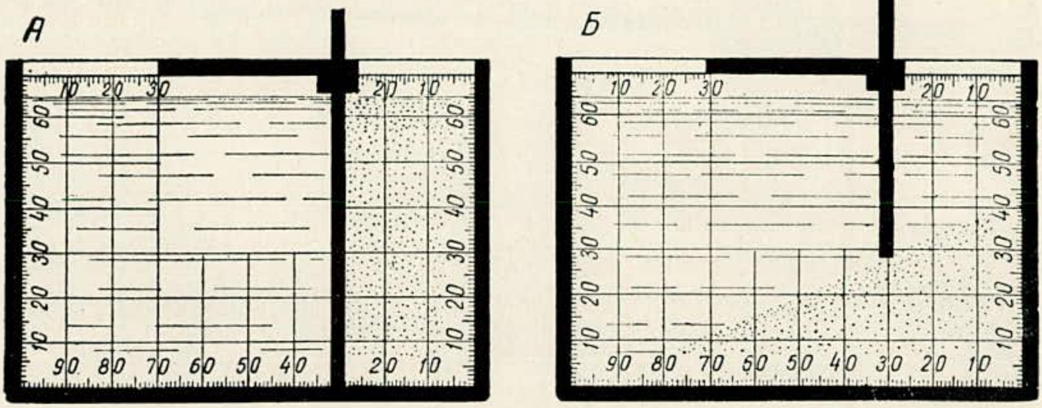

FIGURE 5
Type-9 field laboratory device for determining the angle of repose of sand.

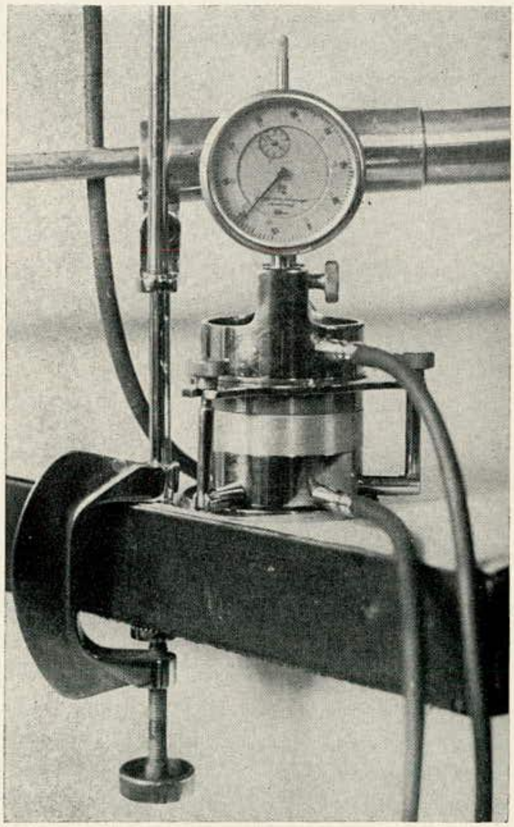

FIGURE 6
Compression device of Type-9 field laboratory in working condition.

tion; the height of the oven when folded is 7 cm. The oven allows 50 samples of soil to be dried simultaneously on the site at different circuit voltages and with automatic temperature regulation.

The high accuracy of operation is due to some constructional features of the sampling equipment (shape of sampling tube, guides, smooth operation of the lever arrangement for pressing the sampling tube into the soil, etc.). It is also due to the fact that the determination of the volume weight and the preliminary weighing for determining the natural moisture content are carried out directly on the site, avoiding a number of intermediate operations that are necessary with the usual method and which exert an undesirable effect on the change of the natural condition of the soil samples by introducing additional errors.

In comparing the degrees of accuracy and the time required with different methods of control sampling of block samples of the same undisturbed soil, it has been found that the maximum difference between such parallel (control) determinations of the volume weight varies within limits:

(a) 2 to 6 per cent for the method of large cylinders (D = 125 mm).

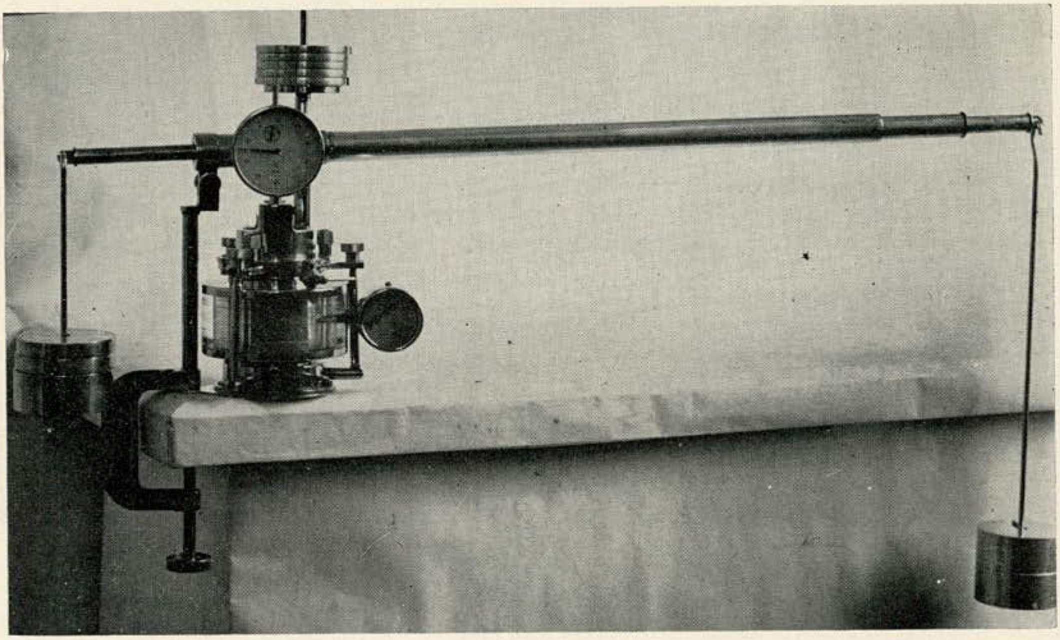

FIGURE 7
General view of device for field investigation of soils when sample is subjected to compression on all sides (Type-9 field laboratory).

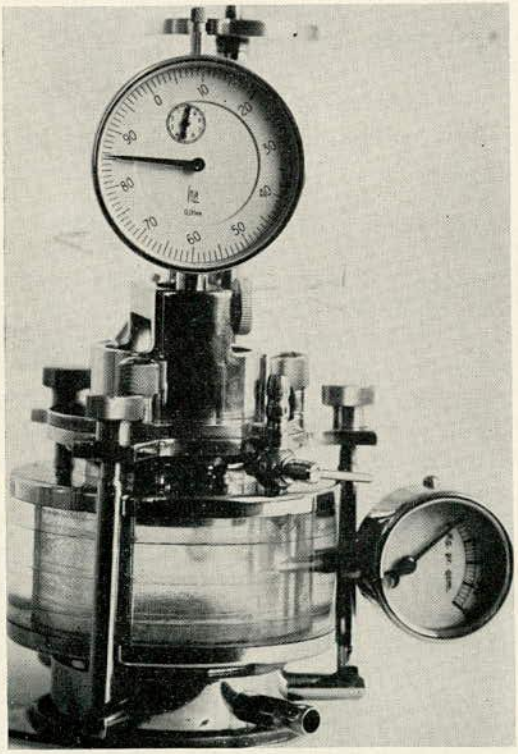

FIGURE 8
Working part of device shown in Fig. 7.

(b) 5 to 10 per cent for the mercury displacement method.

(c) 0.5 to 1 per cent in using the field laboratory equipment.

The time required for the selection and packing of soil block samples with undisturbed structure and the determination of volume weight is:

(a) 190 to 200 minutes for the method of large cylinders.

(b) 90 to 110 minutes for the mercury displacement method.

(c) 8 to 10 minutes using the field laboratory equipment.

Some data on the economical efficiency achieved by using this equipment will be given here. The cost of selection of one soil block sample by the conventional methods varies between 25.5 and 62 roubles, while using the field laboratory equipment would reduce the cost to but one rouble. Hence, if 20 block samples a day are taken, i.e., when the equipment is made use of only to 20-25 per cent of its full efficiency, the cost of soil sampling work would be reduced 500 roubles in one day, or 125,000 roubles during a year.

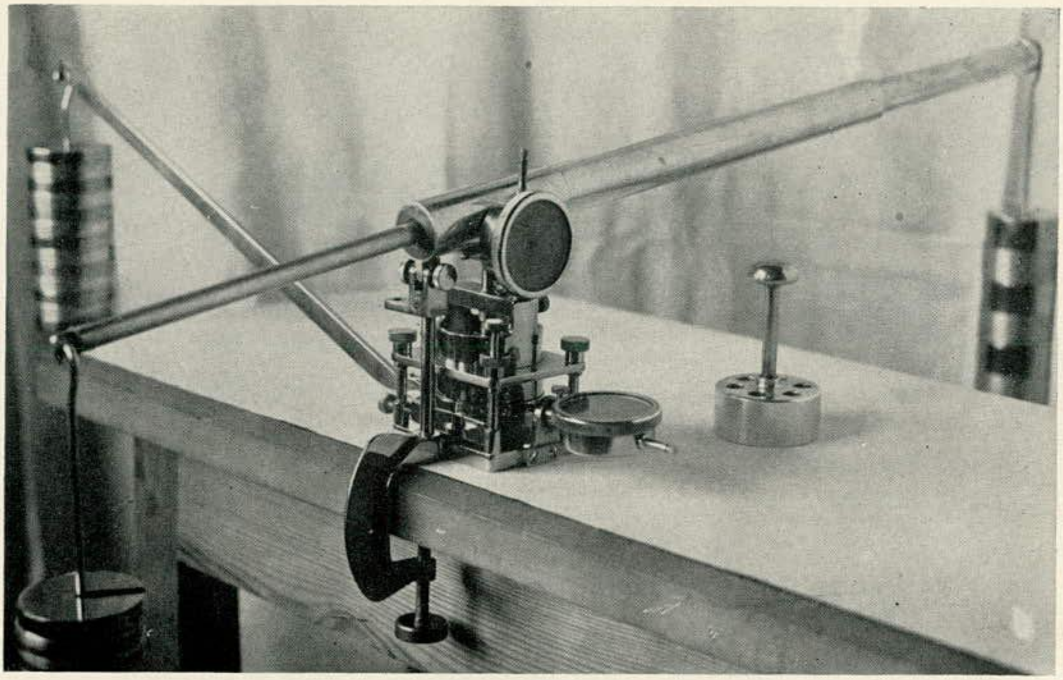

FIGURE 9
General view of device for field shear tests of soils (Type-9 field laboratory).

The determination of the basic physical properties of soil samples of undisturbed structure and natural moisture content by means of a field laboratory is actually reduced to simple operations: sampling blocks of soil and weighing them before and after drying. The other basic physical properties of the soil (volume, weight, natural moisture content, porosity, coefficients of porosity, density) are further determined by very simple mathematical computations.

The compressive properties of cohesive soils, the granulometric composition and density of loose soils, different factors characterizing the moisture content, and other characteristics are determined by means of devices specially designed for work on the site and contained in the field laboratory equipment. The work is carried out following the procedure described in a manual on the use of field laboratories.*

A special device for shear tests of soils directly on the site has been developed by the author and included in the Type-9 field laboratory as an integral part of the equipment. (See Figures 9, 10, 11.)

The device is of the single-shear type. The main feature of the device is the plane which separates the stationary part of the tube with the soil from the movable part and in which shearing of the soil sample is effected. This plane is not designed parallel to the line of motion of the movable part as in other equipment, but at a small angle α which never exceeds several degrees. The gap which automatically forms during the horizontal motion of the movable part of the device widens while the test is being carried out. In this manner it is possible to eliminate completely the sliding friction of the metal. At the same time any possibility is here avoided of wedge action by particles of sandy soil which considerably increases the accuracy of testing.

Another device has been developed to be used with the Type-9 field laboratory for accelerated control of the quality of soil compaction in different earthworks. This device, which has been thoroughly tested in laboratory work and in field investigations,

* I. M. Litvinov, Field Investigation of Soils, Second Edition, Revised, Coal Mining Publishing House (UGLETEKHISDAT), 1954.

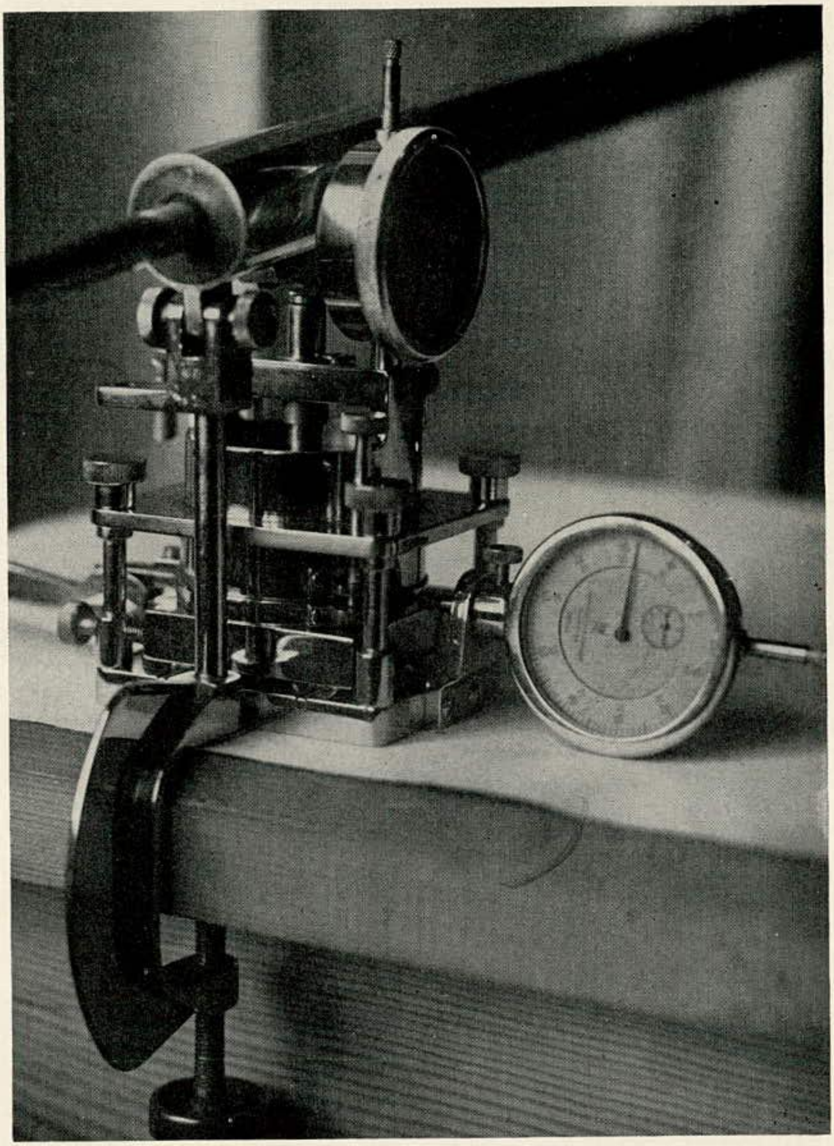

FIGURE 10
Working part of the device shown in Fig. 9.

can be used with success in hydraulic engineering, road construction, etc. The volume weight, moisture content, porosity and other physical soil characteristics can be determined quickly and with high accuracy and without preliminary drying out of the samples. In addition, the device permits the specific weight of the solid phase of soil to be determined directly in the field, with very high accuracy just as it could be done in stationary laboratories.

The operation of the device is based on the hydrostatical weighing of soil samples in water. The total weight of the device with case is 2.2 kg.

The author has also developed a set of vibrational equipment of original design for accelerated and dynamical investigations of soils, which can be used for the determination of the compressive properties, shear strength and density. The main part of the equipment is a high-frequency vibrator

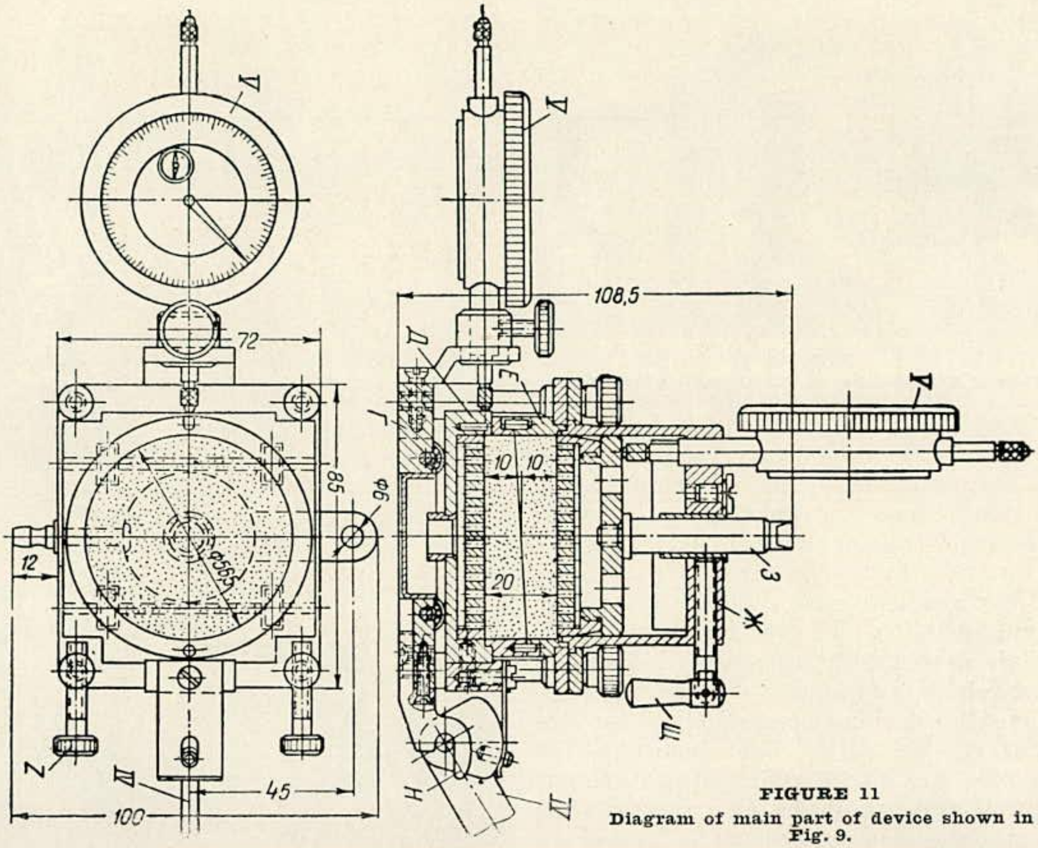

FIGURE 11
Diagram of main part of device shown in Fig. 9.

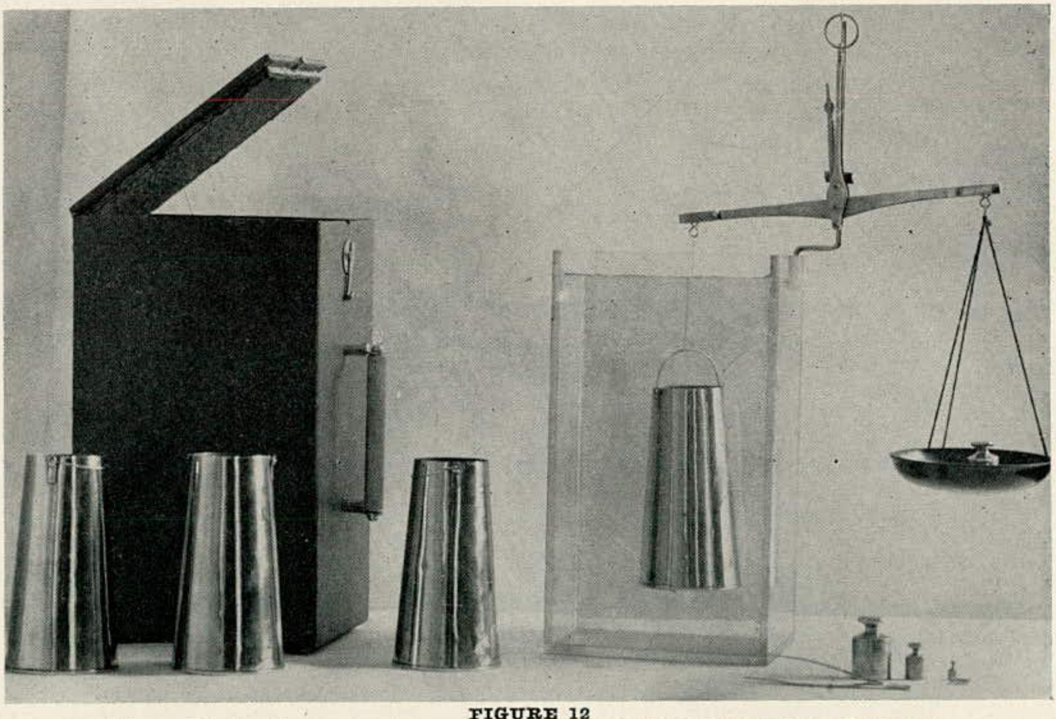

FIGURE 12
View of device for accelerated quality control of soil consolidation in earthworks.

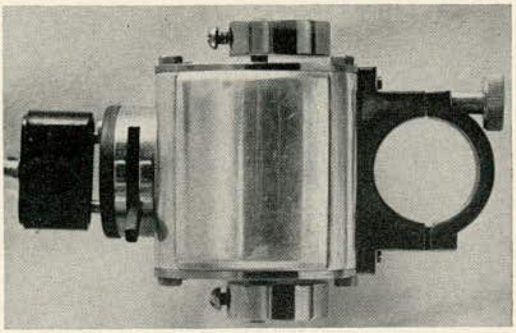

FIGURE 13
View of Type-9-B set of vibrational equipment for compression and shear test devices.

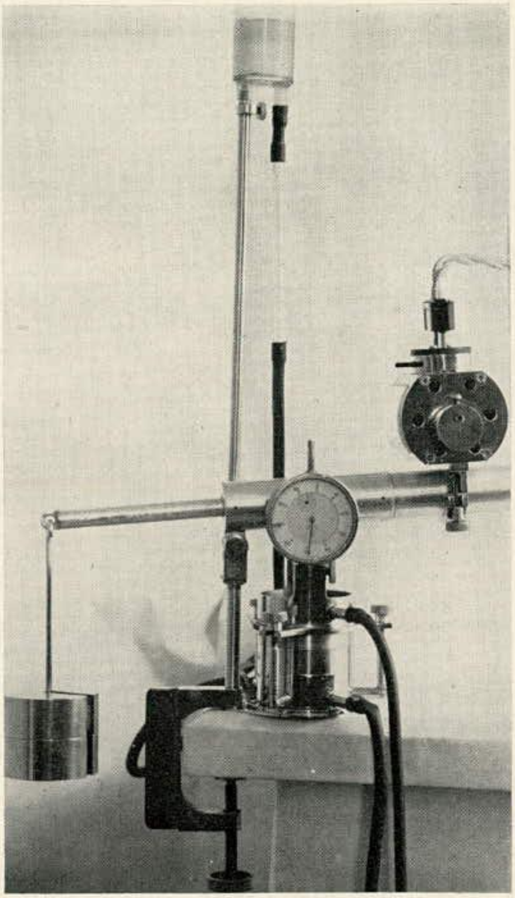

FIGURE 14
Type-9-B set of vibrational equipment during compression tests.

(Figs. 13, 14), mounted on the loading lever of the compression device. Through the loading lever vibrations of given direction, frequency and amplitude are induced by the vibrator in the loaded member.

The weight and size of the counterweights symmetrically mounted along the motor shaft, as well as the amplitude of vibration, are determined in such a manner that, after the settlement has been stabilized for any stage of the statical load applied to the sample, starting the vibrator and its subsequent operation during a given interval of time do not cause additional increase in the settlement. The vibrational equipment is also applicable to dynamical investigation of soils.

The field laboratories are widely used for the investigation of the building properties of soils in housing, civil engineering, industrial building, road and railway construction, hydraulic engineering, etc., in many countries. These devices are of special value in regions remote from the stationary well-equipped laboratories.

Use of the Vibratory Method for Sinking Piles and Pile Shells in Bridge Construction in the USSR

M. M. LEVKIN, *Chief Engineer,*
Department of Large Bridges, Moscow

The following vibratory drivers were used during the past six to eight years in the bridge building practice of the USSR for sinking piles and pile shells:

VIBRATORY DRIVER VP-1 *(Fig. 1)*

This is a mechanism of single frequency action which has the following structural parts:

1. The body of the vibratory machine consisting of a steel box with outside guide rollers and with loops at the upper corners for raising the vibrator.

2. Four working shafts connected to the body of the vibrator by bearings; eccentric weights are attached to these shafts.

3. A system of cylindrical gears which transmit the rotational movement of the electromotor to the working shafts.

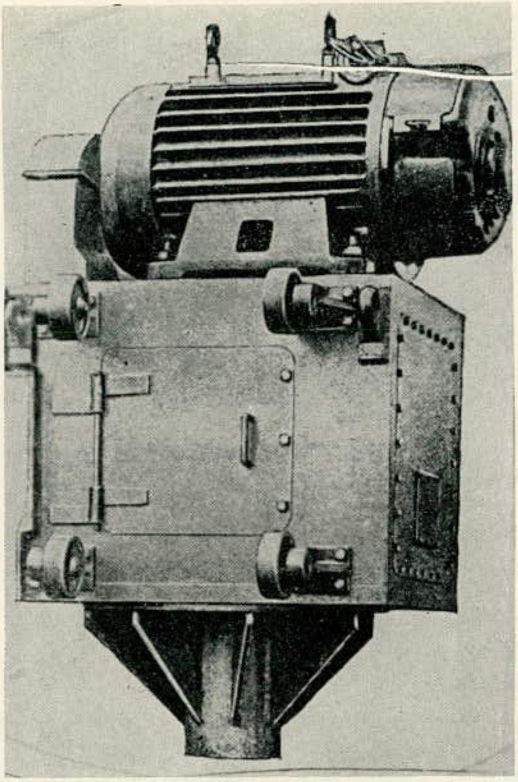

FIGURE 1

4. A conical base for the attachment of the pile to the vibrator.
5. Electromotor.

The dimensions of the vibratory driver VP-1 are: in plan (including the rollers and the loops) 1,300 x 1,240 mm (= 51.2 x 48.8 in.); the height without the base is 1,650 mm (= 65 in.). The height of the base is 450 mm (= 17.7 in.). The weight of the vibratory driver is 4.5 metric tons (= 4.9 long tons). The static moment of the eccentric weights equals 10,000 kg.-cm. (= 8,700 in.-lbs.). The frequency of the oscillations depends on the revolutions of the shafts and equals 420 rpm.

This develops a maximum exciting force of 19 metric tons (= 21 long tons). The amplitude of the vibratory driver in neutral gear equals two centimeters (= 0.79 in.). The power of the electric motor equals 60 kilowatts.

The vibratory driver VP-1 is used in bridge building practice for sinking into the soil piles the critical resistance of which does not exceed 100 metric tons (= 110 long tons).

The attachment of the vibratory driver VP-1 to the head of a pile is made with the help of a conical device which consists of a steel cone and its cone base. The cone base is attached by bolts to the bottom of the vibrator and the cone is attached to the head of the pile. The attachment of the plate of the cone to the pile is usually achieved by means of a steel cover placed on the head of the pile.

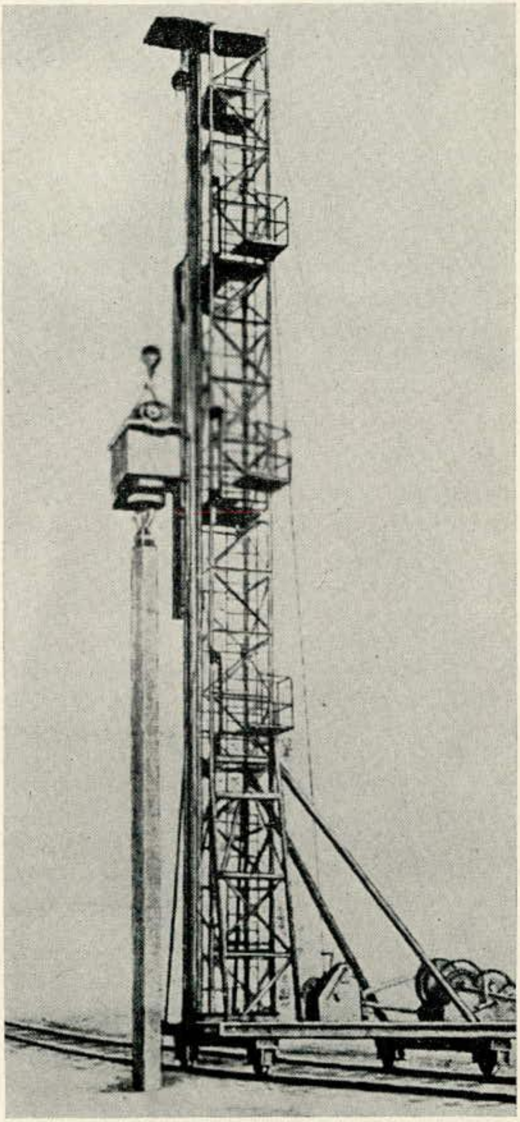

FIGURE 2

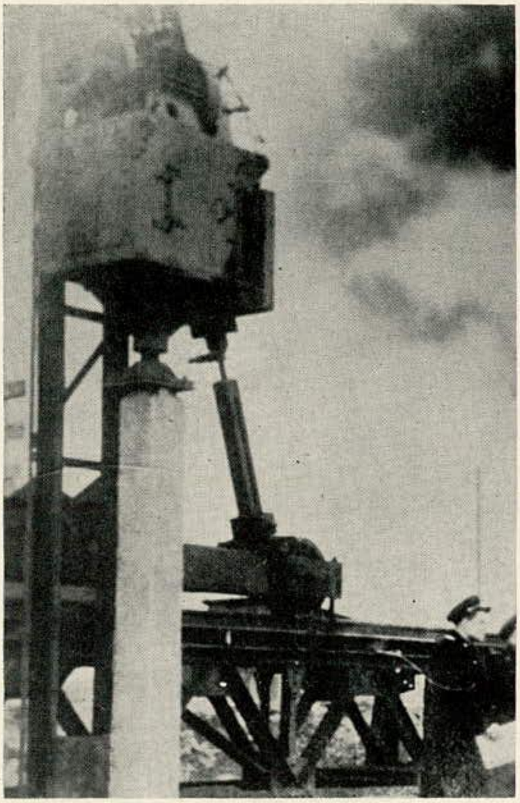

FIGURE 3

The cover consists of two plates with four short shafts welded onto them. The shafts are threaded at their ends. The plates are pressed against the pile on two sides by horizontal tension bolts. The upper sides of these plates are in contact with steel projections welded onto the steel reinforcement near the head of the pile and concreted with it.

The sinking of reinforced concrete piles by a vibratory driver VP-1 with the help of a pile driving frame is shown *(Figs. 2, 3)* and with the help of a crane *(Figs. 4, 5)*.

VIBRATORY DRIVER VP-2

This is intended for sinking into the soil light piles, the critical resistance of which does not exceed 50 metric tons (=55 long tons); also for sinking steel and wooden sheet piles.

The overall dimensions are 950 x 950 x 1,270 mm (= 37.4 x 37.4 x 50 in.). Weight is 2.1 metric tons (= 2.3 long tons). The static moment of the eccentric weights is 4,000 kg.-cm. (= 3,470 in.-lbs.). The number of revolutions of the shafts is 445 rpm.

The exciting force is 8.4 metric tons (= 9.2 long tons). The amplitude in neutral gear is two centimeters (= 0.79 in.). The power of the electric motor is 22 kw.

VIBRATORY DRIVER VP-3

This is designed to sink into the ground piles with a critical load up to 200 metric tons (= 220 long tons) and also for the sinking of pile shells weighing up to 15 tons. Overall dimensions are 1,560 x 1,500 x 2,000 mm (= 61.4 x 59.0 x 78.6 in.). Weight is 7.5 metric tons (= 8.3 long tons). The static moment of the eccentric weights is 26,300 kg.-cm. (= 22,850 in.-lbs.). The number of revolutions of the shafts is 408

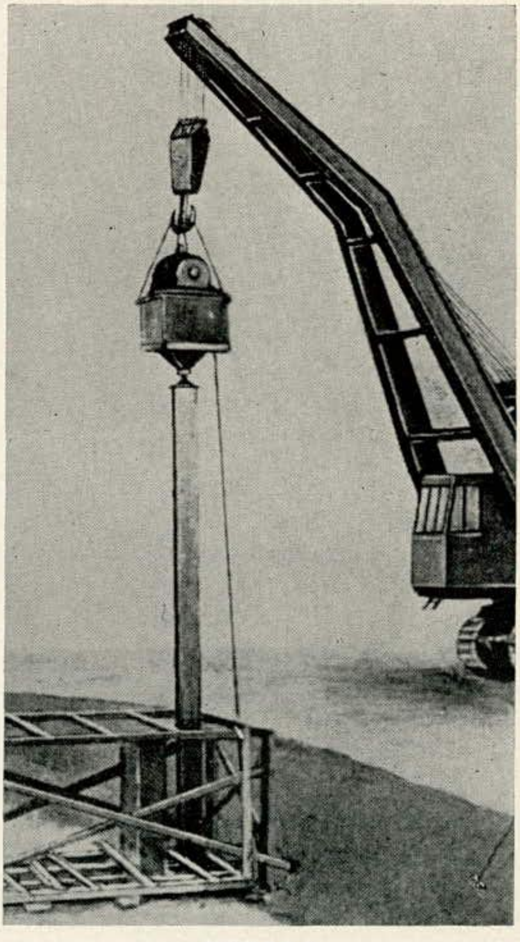

FIGURE 4

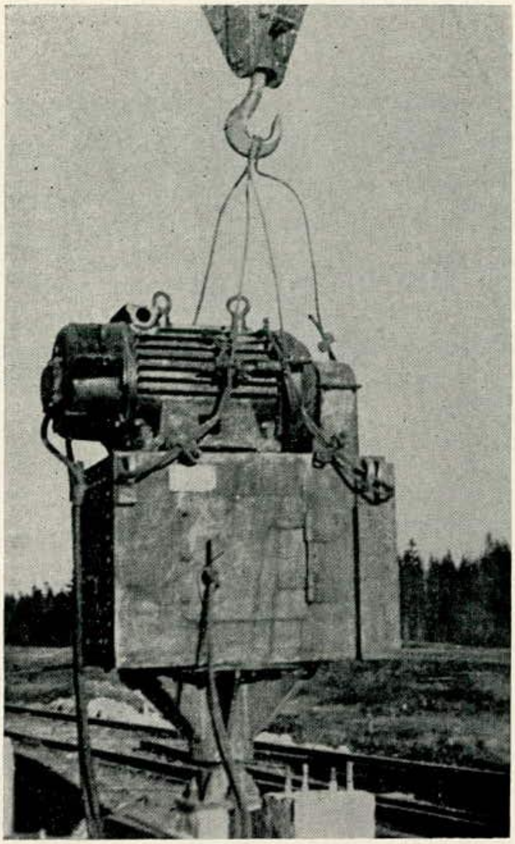

FIGURE 5

VIBRATORY DRIVER VP-160 *(Fig. 10)*

This is intended for sinking into the ground reinforced concrete shells up to five meters (= 19.7 ft) outer diameter to a depth of 30 meters (= 98 ft). The vibratory driver VP-160 is a low frequency 8-shaft vibrator of 2-frequency action. The construction of the vibratory driver permits separate control of the frequency of vibration, the magnitude of the exciting force and of the load moment. Overall dimensions are 1,500 x 1,180 x 3,100 mm (= 59 x 46.4 x 122 in.). Weight is 10.4 metric tons (= 11.5 long tons).

The maximum moment of the eccentric weights is 39,000 kg.-cm. (= 34,000 in.-lbs.). The vibratory driver has three speeds of rotation of its shafts: The first, 800/400 rpm. Exciting force is 44.2 metric tons (= 49 long tons). Amplitude in neutral gear is three centimeters (= 1.18 in.). Power of the electric motor is 100 kw.

Shown *(Figs. 6, 7)* is the sinking of a reinforced concrete shell 15.5 meters (=49 ft) long with an outer diameter of 90 centimeters (= 35.4 in.) and a wall thickness of six centimeters (= 2.36 in.) by a vibratory driver VP-3 with the help of a crane. Also shown *(Fig. 7)* is the ejection of the soil from the interior of the shell by means of an airlift. Soil was sandy gravels.

Shown *(Figs. 8, 9)* is the sinking by a vibrator VP-3 of reinforced concrete shells up to 27 meters (= 88.5 ft) long with an outer diameter of 1.20 meters (= 47.3 in.) and a wall thickness of 10 centimeters (= 4 in.). Soil was silty sand.

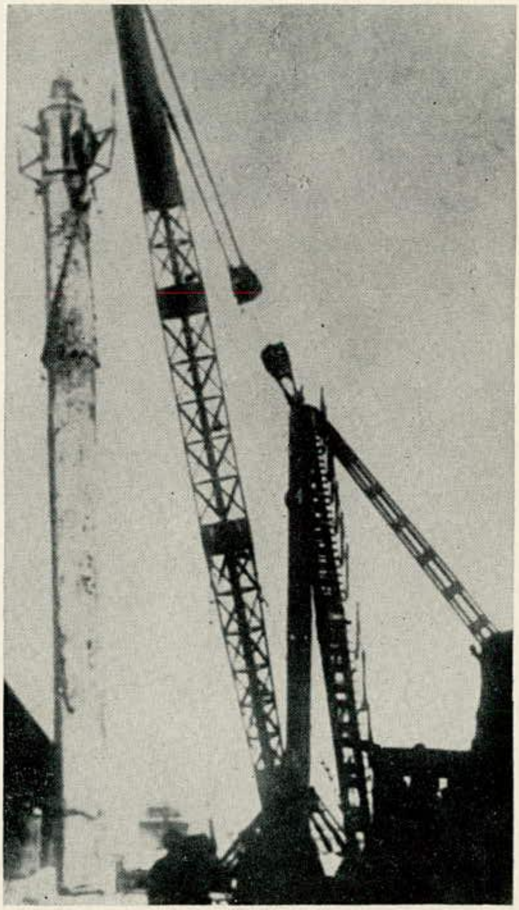

FIGURE 6

FIGURE 7

rpm; second, 900/450 rpm; third, 1,010/505 rpm.

The exciting force is 100 metric tons (= 110 long tons); 130 metric tons (= 144 long tons); 160 metric tons (= 177 long tons). The power of the electric motor is 155 kw. The vibratory driver is adapted for synchronous operation. This vibratory driver has been in production since 1958.

Shown *(Fig. 11)* are two vibratory drivers VP-160 joined together for synchronous operation. On one of them the lid has been removed and one can see the gears and the shafts.

Shown *(Fig. 12)* is the sinking by a vibratory driver VP-160 of a reinforced concrete shell 18 meters (= 59 ft) long with a diameter of 1.60 meters (= 52.5 in.) and with a wall thickness of 10 centimeters (= 4 in.) into sandy clay soils.

Shown *(Fig. 13)* are two paired vibratory drivers VP-160 mounted on a shell of a 3-meter (= 9.8 ft) diameter.

For the purpose of sinking shells up to six meters (= 19.7 ft) diameter to a depth up to 40 meters (131 ft), a vibratory driver VP-250 was to be produced during 1959.

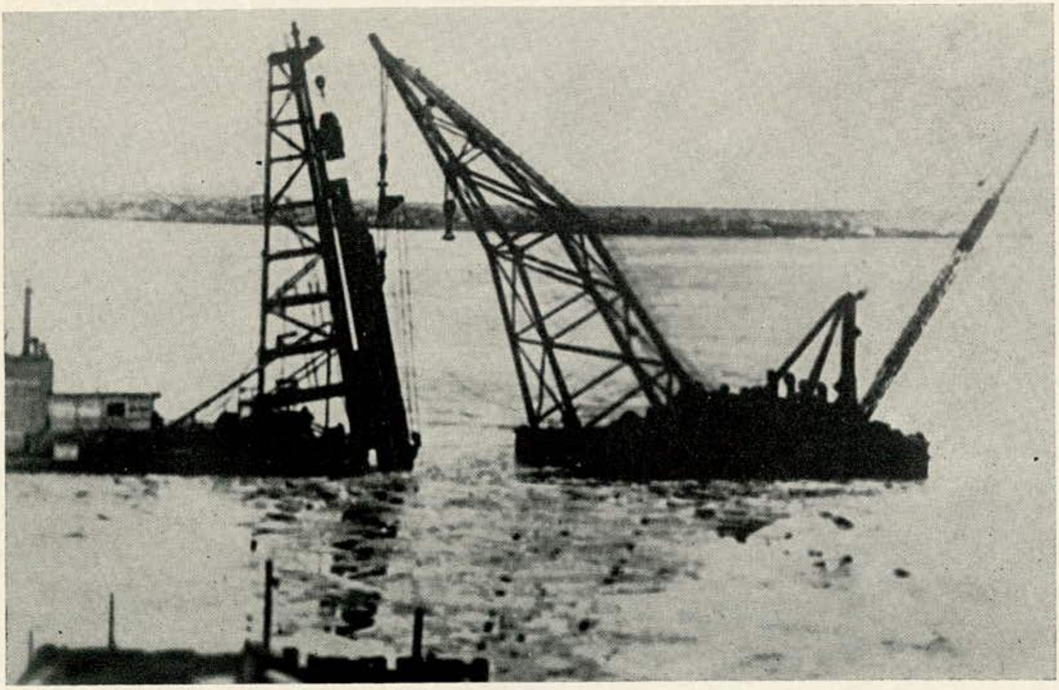

FIGURE 8

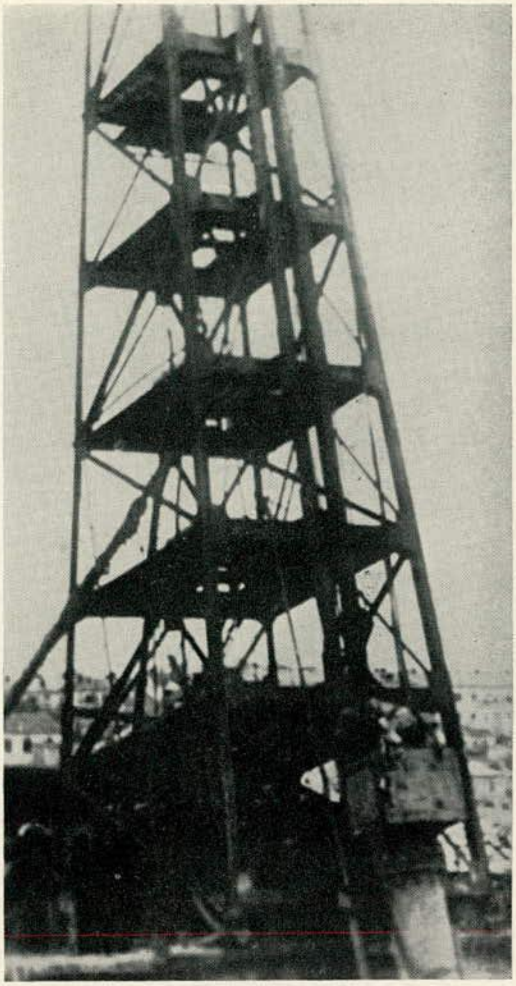

FIGURE 9

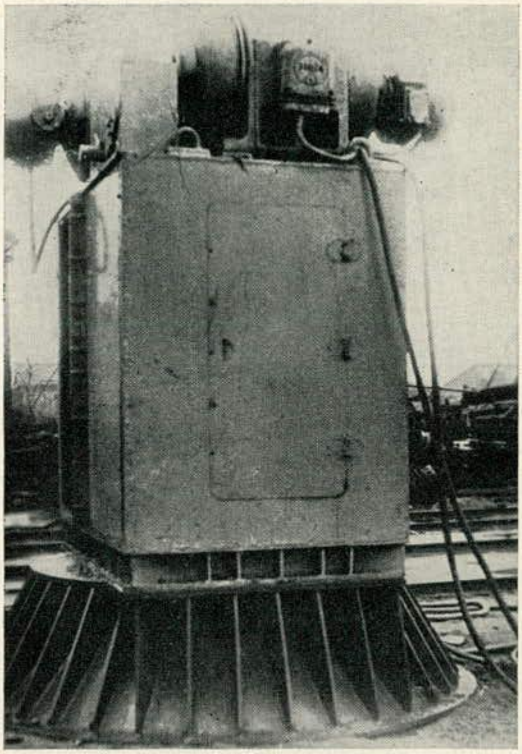

FIGURE 10

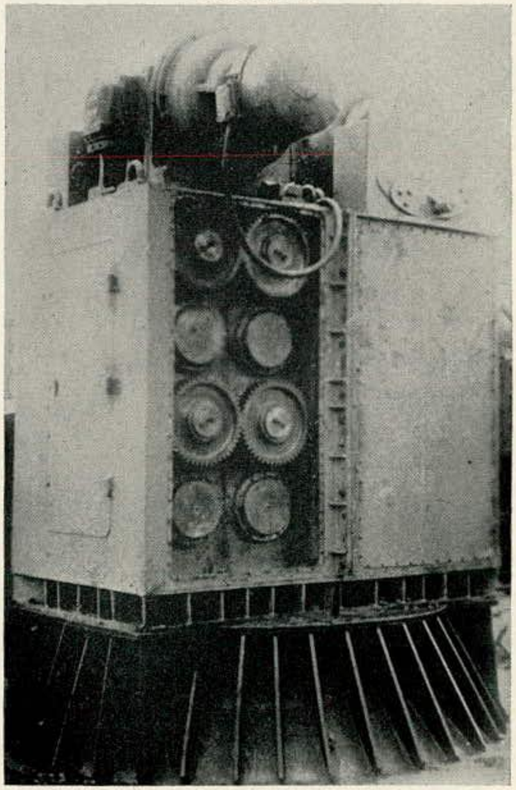

FIGURE 11

It would have a maximum value of the exciting force equal to 250 metric tons (= 275 long tons) and it would also permit variations of the value of the load moment and of the frequency of oscillations. The possibility of synchronous action of these vibratory drivers is also foreseen.

In connection with the abandonment of deep massive foundations for bridge piers and the adoption of composite reinforced concrete shells during the coming year on the construction sites of several large bridges, a large number of composite reinforced concrete shells from 0.6 to 5 meters (= from 1.97 to 16.3 ft) are to be sunk by

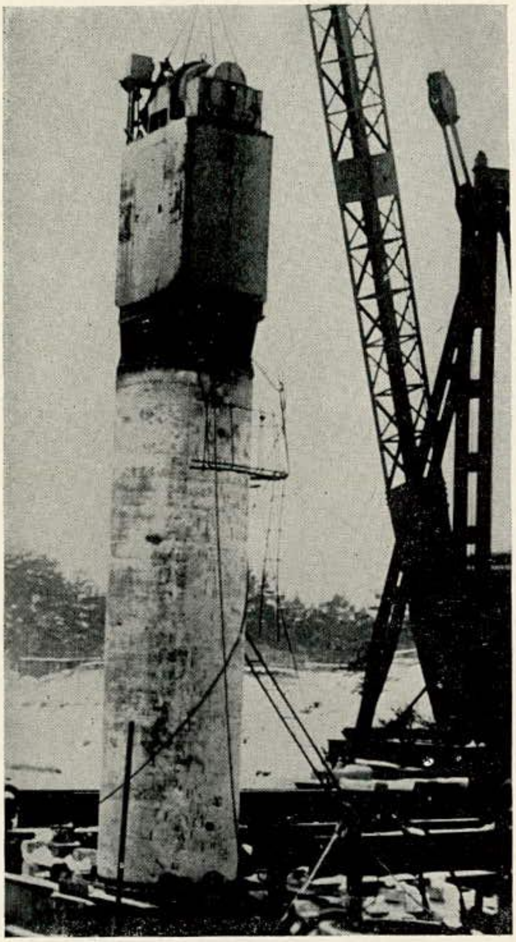

FIGURE 12

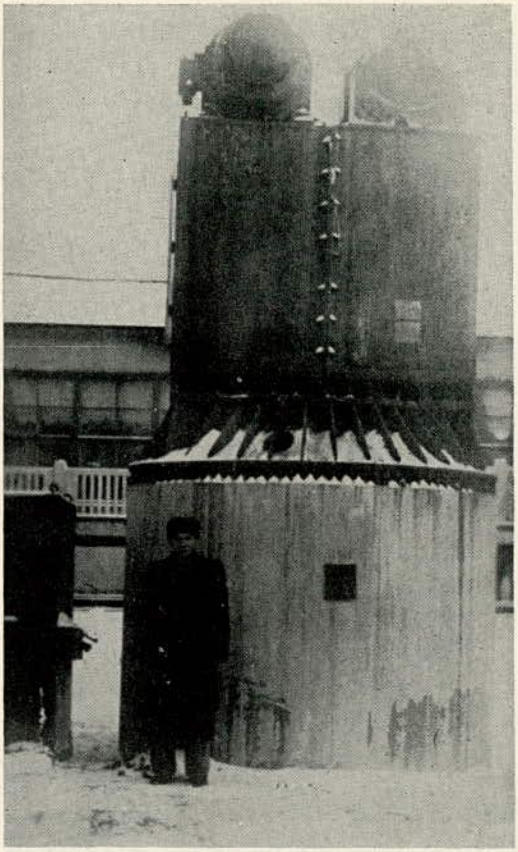

FIGURE 13

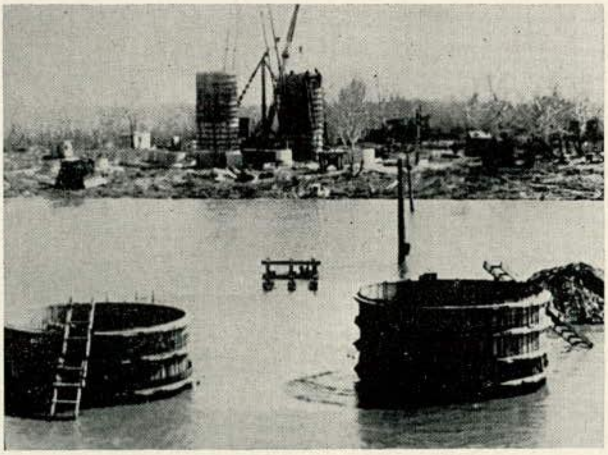

FIGURE 14

means of vibratory drivers. Shown *(Figs. 14, 15)* is the manufacture of reinforced concrete shells of 5.0 meters (= 16.3 ft) diameter with a wall thickness of 14 centimeters (= 5.5 in.) on a motor road bridge under construction. They are being sunk into sand with fine gravel to a depth of 22 meters (= 72 ft).

FIGURE 15

On the Development of Soil Mechanics in the USSR

Nikolai A. Tsytovich

The present communication is devoted to the development of the basic ideas of modern soil mechanics in the work of Russian and Soviet scientists.

The scientists and the engineers of the USSR attach considerable importance to the solution of problems in the field of soil mechanics, since important territories of the Soviet Union are covered by loose deposits of mineral soils of considerable depth. The erection of structures on such soils requires specific knowledge of soil mechanics.

This is all the more important since during the last decades many industrial and housing structures are being erected in the Soviet Union which often structurally are statically indeterminate and which are very sensitive to uneven settlements. A forecast of the magnitude of such settlements can be made only through the use of solutions provided by soil mechanics.

All this compelled Russian scientists a long time ago to devote their studies to the development of problems of the theory of bases of structures (publications of V. I. Kourdyumov, 1889 and others) and to their practical applications to foundation engineering. It is significant that, for instance, a text on "Bases and Foundations" by V. M. Karlovich *(Ref. 1)* was published in the Russian language in 1869 and a systematic text on soil mechanics appeared in 1934 *(Ref. 3)*.

In the following we shall consider:

1. The development of problems of the theory of soil mechanics in the work of Soviet scientists.
2. The development of the fundamentals of the mechanics of separate regional types of soils.
3. The use of solutions provided by soil mechanics for the design of foundations and the performance of earth moving and foundation construction operations.
4. The direction of the latest research in the field of soil mechanics in the USSR.

Problems of the Theory of Soil Mechanics

The following should be considered as basic problems in the field of the theory of soil mechanics to the development of which Soviet scientists have devoted studies during the past 40 years: (a) the application of the theory of elasticity to the design of bases; (b) the study of the interaction between structures and their compressed base; (c) the development of general solutions of rigorous theories on the limit state of stress; (d) the theories of consolidation of water-saturated soils, and (e) the problems of the dynamics of soil bases.

A. The first question which arose in connection with the use of the theory of elasticity for the design of bases was: Is the theory of elasticity applicable to soils of bases? As had already been shown in 1916 by B. A. Minayev *(Ref. 4)*, the use of the basic equations of the theory of elasticity for computations related to granular media is most fruitful. N. M. Gersevanov in his publications of 1930-1933 *(Refs. 7, 8)* has shown that the application of the theory of elasticity to clays and to very fine sands is quite as justified as is its application to steel, whereby was proposed for the nonelastic state of soils the criterion of linear deformability *(Ref. 7)*. In 1936, V. A. Florin *(Ref. 9)* established a relationship between the dimensions of plastic zones in the soil and the dimensions of the structure, which determines the conditions of applicability of the solutions of the theory of elasticity to the design of bases.

At the end of the 1920's and at the start of the '30's, the theory of elasticity began to be widely applied in the USSR to the design of natural bases. Here should be pointed out the remarkable work of N. P. Pouzirevsky *(Refs. 5, 6)*, who proposed an original method for the use of the theory of elasticity which permitted him:

To develop a general theory of the stresses in earthy soils and in particular to give a solution for the problems of the magnitude of pressures in the base of a structure corresponding to the beginning of the appearance of the plastic zones under the edge of the foundation (i.e., he obtained in 1929 an equation for the critical edge load, an identical expression for which was also published in a somewhat different form by O. K. Froehlich in 1934).

To justify an analytical expression for the slip surface in slopes later used for the computations of their stability.

To give equations for lateral earth pressures against retaining structures for various cases of their loading, as well as a number of other solutions.

At about the same time, N. M. Gersevanov *(Ref. 7)* proposed to consider three basic phases of the stressed state of soils under foundations: the phase of consolidation, the phase of displacements (plastic flow) and the phase of squeezing out (progressive flow after N. A. Tsytovich). By a procedure different from the solution of N. P. Pouzirevsky, he obtained for the edge loading the value corresponding to the beginning of the appearance of the displacement phase in soils, which value in practice equals the permissible pressure on the soil.

B. Studies of the interaction between structures and their compressed bases have received much attention from Soviet scientists. Efforts were directed mainly towards the development of general theoretical solutions. Here, first of all, should be noted the work of Pouzirevsky *(Refs. 5, 11)* and of Academician A. N. Krylov *(Ref. 10)* who by developing a method of initial parameters reduced the complex fourth order differential equation for the design of beams on a local (Winklerian) base to a solution of two equations with two unknowns.

This permitted the development of a very effective method for the determination of all design values. As was shown by later investigations (N. A. Tsytovich and others), the method of the design of beams and of slabs on a local elastic base (developed on the assumption of direct proportionality between the pressure and the elastic settlement only at the place of the application of the load),* can be successfully used only in the case of highly compressible soils and of a small thickness of the deposits underlain by incompressible layers such as rock.

During the further development of the work of USSR scientists, the model of a locally compressed base was improved: First, in the direction of a more precise evaluation of the elasticity of the base (the method of M. M. Filonenko-Borodich according to which the elastic base may be represented by a localized Winklerian base superposed on a membrane stretched in all directions); and second, in the direction of the introduction of two coefficients to characterize the compressed base (i.e., of coefficients of elastic compression and of elastic shear or slide, after P. L. Pasternak) *(Refs. 12, 13)*.

However, the model of a locally elastic base in many respects did not satisfy Soviet scientists in spite of its improvements, since it did not permit the determination of soil deformations at some distance away from the place of load application, but mainly because its characteristic coefficients were not constant. Therefore, they started a long time ago the direct use in its pure form of the theory of elasticity for the design of foundations on a so-called elastic half space.

Here one should note apparently the first publication in that field, the work of G. E. Proctor *(Ref. 14)*; and then the publications of N. M. Gersevanov and I. A. Macheret *(Ref. 15)*; V. A. Florin *(Refs. 16, 17)*, as well as B. N. Zhemochkin, S. S. Davidov, V. Z. Vlassov, B. I. Kloubin and others *(see, e.g., Ref. 18)* and a generalized summary publication by M. I. Gorbounov-Possadov *(Ref. 19)*. In this latter publication were given the results of the research of Gorbounov-Possadov as well as of other Soviet authors with numerous tables for the rapid and precise design of beams and slabs on an elastic or linearly-deformable half space.

As was shown by special discussion of the methods of design of beams and of slabs on a compressible base *(Ref. 18)*, the method of an elastic half space, i.e., the method of general elastic deformations, is more applicable to compact and hard soils which can be assumed uniform to a sufficient depth below the lower surface of the foundations.

It is also interesting to note that a general method for the solution of the so-called contact problem of distribution of pressures along the lower surface of a foundation under conditions of a two-dimensional problem for foundations of any rigidity and for any load on the foundation as well as beyond its periphery was published in 1936 by Prof. Florin *(Ref. 16)*. This paper also considered the effect of tangential stresses along the lower surface of the foundation. A number of partial problem solutions were published in 1949 by I. I. Steuerman *(Ref. 20)*.

The further development of this type of problems of the theory of soil mechanics is taking place along the lines of improvement of the design models of the soil and the development of solutions of mixed problems of the theory of elasticity and of the theory of plasticity *(see, e.g., Ref. 21)*.

C. The development of a general theory for the limit stressed condition of soils both in the formulation of the problem itself as well as in the development of mathematically rigorous solutions belongs mainly to Soviet scientists. Thus in 1939-1942 were published the remarkable works of V. V. Sokolovsky *(Refs. 22, 23)*, in which were given the general analytical solution of a two-dimensional problem of the theory of limit equilibrium for uniform and for layered, isotropic and anisotropic granular media and, specifically, a precise determination of the value of lateral pressures of soils against retaining structures, of the stability of soils in the bases of structures, of the critical height of slopes and of their equivalent stable form, etc.

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* The hypothesis concerning direct proportionality of pressures and local settlement of the soil was advanced as early as 1801 by the Russian Academician N. I. Fouss.

A grapho-analytical solution of the two-dimensional problem of limit equilibrium for granular media was studied by S. S. Gouloushkevich *(Ref. 24)*. The three-dimensional problem of the theory of the limit stressed state of soils has been elucidated in the publications of V. G. Berezantzev *(see, e.g., Ref. 25)*.

The latest studies of Soviet scientists concerning the theory of the limit stressed state of soils are devoted to the consideration of various boundary conditions, to the greater precision of the basic equations for the limit equilibrium of soils, to the study of the strength of bases under deep foundations and to other problems resulting from the practice of the design of structures.

D. The theory of consolidation of water-saturated soils has received wide development after publication of the works of K. Terzaghi (1925, and others). The studies of Soviet scientists in this branch of soil mechanics were directed towards obtaining more rigorous solutions for the linear problem of consolidation, to the formulation of the two- and three-dimensional problem of consolidation and to the development of effective methods of solutions for different boundary conditions and for varying values of the coefficients characterizing the consolidation.

Thus, one should specially mention here the work (1931-1948) of N. M. Gersevanov *(Refs. 26, 27)*, in which the problem of consolidation of a layer of water-saturated soil under a continuous load was studied in detail, equations were formulated of consolidation of a uniform and isotropic soil mass for the cases of a two-dimensional and of a three-dimensional problem, and a solution of the problem of initial stresses when loading a soil mass by an instantaneous strip load was obtained. A solution of the problem of swelling of a layer of clay soil was published by S. A. Rosa in 1937 *(Ref. 28)*; methods of consideration of the initial gradient of hydraulic head were published by him in 1950 *(Ref. 29)*.

A complete formulation of the two-dimensional and of the three-dimensional problem of the theory of consolidation of an earth mass was published by V. A. Florin in 1938 *(Ref. 30)*. In 1948 *(Ref. 31)* he published numerical methods for the solution of these problems under any special conditions as well as methods for the determination of the initial effective and neutral stresses at any moment in time for the case of uniform and non-uniform anisotropic media under consideration of changes in the coefficient of permeability.

The theory of consolidation of soils received further development in the work of D. E. Polshin *(Ref. 15)* who formulated in a general form the three-dimensional problem of the filtration theory for rectilinear and non-rectilinear consolidation of clay soils; also in the latest publications of V. A. Florin *(Refs. 32, 33)*, who studied the consolidation of clay soils in the presence of an initial hydraulic gradient and of entrapped air, as well as under consideration of the creep of the mineral skeleton of the soil and of a gradual increase of the consolidating zone as the structure was being erected.

E. The problems of dynamics of soil bases were developed by Soviet scientists in two directions. They studied the vibrations of foundations and the conditions of liquefaction of water-saturated loose sands under dynamic action.

In the USSR the work of N. P. Pavliuk, "On the Oscillations of a Solid Body Resting on an Elastic Base," which was published in 1933 *(Ref. 34)*, is rightly considered a fundamental publication concerned with the vibrations of foundations.

An important contribution to the dynamics of bases was made by D. D. Barkan who published the results of many years of studies on the vibrations of foundations and on dynamic properties of soils *(Ref. 35)*, and by O. A. Savinov who developed methods for the determination of elastic characteristics of soils (the coefficients of elastic uniform and nonuniform compression and of elastic horizontal displacement), necessary for the computation of vibrations of foundations. He also published a summarizing study on the design of foundations for machinery *(Ref. 36)*. S. S. Davidov developed the theory of oscillations of soils under the action of a load of short duration.

There are also important studies in the field of the dynamics of soil bases as regards the conditions of liquefaction of loose water-saturated sands. Here should be noted the theoretical and experimental studies stretching over many years of three groups of USSR scientific workers directed by V. A. Florin *(Ref. 37)*, M. N. Goldstein *(Ref. 38)*, and by N. N. Maslov *(Ref. 39)*.

In the work of Profs. Florin, P. L. Ivanov and others, it has been shown that the condition for the liquefaction of water-saturated sands is determined by the relationship between the intensity of the dynamic action, the porosity of the sand and its stress condition, and not just by the value "of the critical porosity during shear," i.e., as a basis the theory of dynamic dislocation of the structure of water-saturated sands. This theory already had been advanced by N. M. Gersevanov (1948).

Maslov and his collaborators based their studies on the so-called "filtration theory of dynamic stability of water-saturated sands" which is based on the study of the hydraulic heads which develop in water-saturated sands under dynamic action.

Extensive studies for the precision of the concept of "critical porosity" and of the methods for its determination, as well as for the study of the conditions of liquefaction of sands, have been carried out by Goldstein and his collaborators *(Ref. 38)*.

Development of Fundamentals of the Mechanics of Separate Regional Types of Soils

In separate regions of the USSR occur extensive deposits of special types of soils, mainly of structural types, the properties of which differ appreciably from the properties of ordinary soils. The erection of structures on such soils without consideration of their special features leads to inadmissible deformations which frequently cause complete collapse.

A number of Soviet scientists devoted their work to the study of regional types of structural soils, developing the fundamentals of the mechanics of these types of soils.

As regional types of soil we consider:

(a) Mud and peaty deposits (organic-mineral deposits).

(b) Loesses and loessial soils (macroporous deposits).

(c) Frozen and permanently frozen soils (cryogenic deposits).

During recent decades as a result of the efforts of Soviet scientists the fundamentals of the mechanics of separate regional types of soils have been formulated and methods for the successful erection thereon of major structures have been developed.

A. Problems of mechanics of mud-silts have been treated in a series of publications of the Institute of Foundations in Moscow (the work of D. E. Polshin and others) and of the Hydro-Energo-Project in Leningrad (the work of S. A. Rosa and others). In these publications it was shown that organic silts under low pressures not exceeding their stuctural strength have one set of properties (neither their compressibility nor their resistance to shear in practice depend on their natural water content), whereas under pressures which produce a destruction of their structural strength they become excessively compressible and unstable soil formations *(e.g., Ref. 40)*.

A series of investigations by Leningrad scientists *(see, e.g., Ref. 41)* has been devoted to the study of peaty soils and of peats as bases for structures and much attention has been given to this problem by the scientists of Byelo-Russia *(see, e.g., Ref. 42)*. As a result of the study of the laws of deformation of peaty soils the values of limit deformations of civil and industrial structures during their erection on peaty soils have been outlined and methods of erection and necessary structural measures have been developed.

B. The special features of loess and loessial soils, such as settlements due to their collapse when wetted under load, came to the attention of Soviet scientists in the 1930's when it became necessary to erect a number of industrial buildings on loessial soils.

Here should be noted the studies of U. M. Abelev *(Refs. 43, 44)*, who developed the fundamentals of the mechanics of loessial macroporous soils; the studies of N. I. Denissov *(Ref. 45)*, who studied the nature of the sudden settlements of this type of soil; and the newest work of M. N. Goldstein *(Ref. 46)* and of G. M. Lomize *(Ref. 47)*, which showed the dependence of such settlements on the stressed state of soils and the simultaneous occurrence with vertical settlements of macroporous soils; also of lateral squeezing out of masses of soil with remolded structure when such soils were wetted under load.

C. The problem of the study of frozen and of eternally frozen soils and of the conditions of construction thereon is of particular importance for peaceful construction in the USSR, since approximately 47 per cent of its territory is located within the zones of permanently frozen soils. The erection of structures on these soils is extremely complicated without appropriate measures. Thus, the mass occurrence of deformations of structures erected on permanently frozen soils is well-known, both as a result of the thawing of frozen soils under heated structures and as a result of frost-heaving foundations.

Soviet scientists began the study of research problems of permanently frozen soils a long time ago. One should note the well-known publication of M. I. Soumgin, "Eternal Freezing of Soils Within the Limits of the USSR." *(Ref. 48)*, and the studies of N. A. Tsytovich *(Refs. 49, 50, 51, 52)*, who, on the basis of a synthesis of numerous experimental studies, formulated the fundamental principles of the mechanics of frozen soils on which was based the development of methods of stable erection of structures on permanently frozen soils.

Then there are the studies related to transport construction on permanently frozen soils of M. N. Goldstein *(Ref. 53)*; the studies of S. S. Vialov on the rheology of frozen soils *(Refs. 54, 55)*; the studies of the Institute of Foundations, and others.

Present studies on the mechanics of frozen soils are directed towards the creation of structural mechanics of frozen soils and the development of problems of rheology of ice and of frozen soils.

Utilization of Soil Mechanics for Design of Foundations and Performance of Earth and Foundation Work

On the basis of the development of the theory of soil mechanics and of the development of a series of concrete solutions of its problems and of the verification of the results in nature, it proved possible to elaborate a very progressive method for the design of foundations according to the limit states of the soil bases. This method produces appreciable economies *(Refs. 56, 57)*, and according to Soviet building codes *(Ref. 58)* has been compulsory for all design organizations since 1955.

This method is based on the rule that the design (i.e., the forecast) settlement or the differential settlement of separate parts of the structure must be smaller than limit values established on the basis of direct observations of foundation settlements and of the deformations of structures.

The development of soil mechanics and of the dynamics of bases in particular permitted Soviet scientists, D. D. Barkan and others, to develop and to use with success the vibratory method of driving piles, sheet piles and sampling tubes into granular and into plastic cohesive soils *(see, e.g., Ref. 60)*.

During foundation excavations the problems of water-level lowering and of the use of artificial waterproofing barriers of frozen soils acquire considerable importance. These questions also received attention in the studies of specialists of the Soviet Union, notably those of G. M. Marioupolsky in the Institute of Foundations concerning water-level lowering on construction sites *(Ref. 61)*; N. G. Troupak *(Ref. 62)*, and H. R. Hakimov *(Ref. 63)* concerning the theory and the practice of artificial freezing of soils for construction purposes.

As methods of improvement of properties of weak soils, the following should be mentioned: The original method of artificial compression of clay soils by lowering the head of ground water in underlying sands (method of M. E. Knorre) *(Ref. 64)*, and the methods of chemical and of electrochemical stabilization of soils developed by B. A. Rzhanitzin *(Ref. 65)*, as well as the method of electro-osmotic drainage and water-level lowering in soils clarified in publications of G. M. Lomize *(see, e.g., Ref. 66)* and the method of thermic stabilization of soils with sand used (I. M. Litvinov and others) for the prevention of sudden settlements of loessial soils *(Ref. 67)*.

As main directions of new studies the following can be considered:

Further development of general solutions of the theory of soil mechanics, both on the basis of a more precise formulation of the fundamental physical assumptions using for that purpose refined methods of physical experiments, and on the basis of results of direct observations in nature and on special experimental structures;

The development of the fundamentals of the statistical mechanics of soils, a successful start on which has been provided by the studies of G. I. Pokrovsky *(see, e.g., Ref. 68)*, and the development of the fundamental laws of structural mechanics of soils which is used with success for the solution of specific problems concerned with the erection of structures on structural types of soils such as organic muds and silts, peaty, loessial, permanently frozen and other similar soils *(see, e.g., Ref. 69)*;

The development of methods for a more complete utilization of properties of soils (e.g., during the design of foundations according to the limit states of soil bases) and of methods of alteration of their properties in the desired directions.

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NOTE: All of the above references unless otherwise noted are in the Russian language.

Basic Principles of Flexible Pavement Design and Construction in the USSR

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In the USSR, which comprises a vast territory between 35° to 78° North latitude and from 169° West longitude to 19° East longitude, there are widely varying physical conditions. There are regions now being economically developed, for which rational types of road pavements and bases have not as yet been worked out practically. Therefore, the elaboration of methods for estimating the quality of bases for the design and construction of road pavements is of very great importance in the USSR.

To take into account the climatic factors while projecting roads, the territory of the USSR is divided into road climatic zones coinciding approximately with typical landscape and soil zones; namely, the tundra and forest zones, the wet zones, the forest-steppe, transitional zones, the steppe zones, insufficiently wet zones, the semi-desert, dry steppe arid zone, except for mountain regions.

Moreover, within the limits of each road climatic zone there are various types of terrain distinguished by the influence of local moisture sources, character of surface drainage, and peculiarities of the hydrological conditions.

About 15 years ago the method worked out in the Central Highway Scientific-Research Institute (SOJUZDORNII) under the guidance of Prof. N. N. Ivanov began to be used in the Soviet Union for designing roads. This method for determining the thickness of flexible road pavements was based on the following thesis:

1. The stress conditions in the road structure at the beginning of failure are characterized by a definite ultimate value of accumulated vertical displacement, deflection, of the pavement. This increases in the wet period of the year under the repeated action of automobile wheel loads. For roads with high-duty and high-type pavements with high-speed traffic, the ultimate value of accumulated vertical displacement is limited by the allowable degree of smoothness of the riding surface.

It is necessary, however, to consider the possibility of failure of the road pavement before the accumulated deflections allowable for the given pavement are exceeded, with less strict requirements as regards permitted surfacing irregularity and the ease with which it can be restored. In such cases the allowable deflection is limited by the need to insure the required strength of the road structure.

2. The modulus of deformation of the structure (E) depending on the ratio between the unit load (p) acting on the surface of the structure and the total relative deflection (λ) caused by it, is the criterion for determining the resistance to deformation of flexible road pavements in the USSR.

$$E = f\left(\frac{D}{\lambda}\right) \qquad (1)$$

Relative deflection (λ) is a non-dimensional value equal to the ratio between the absolute deflection (ρ) and the diameter of a circle (B) equivalent to the contact area of dual-tire wheels of the design automobile with the road surface and for the subgrade, with the area of load transfer $\left(\lambda = \frac{\rho}{B}\right)$.

The value of the ultimate relative deflection is set by the requirements for the operating qualities of the pavement. On the basis of numerous investigations, this is accepted in the range from $\lambda = 0.03$ for high-duty road pavements with a very smooth riding surface, to $\lambda = 0.05$ for pavements operating at the safe-ultimate resistance limit, depending on the type of pavement, thickness of the pavement structure, and the relative rigidity of structural layers.

Since the relation between the relative deflection and the load for flexible road pavements is not usually a linear one, the value of the modulus of deformation depends on the relative deflection (λ). When determining the modulus of deformation, its value is calculated at the design figure for relative deflection of the given pavement layer or soil.

In addition to the factors of the area of load contact, thickness and composition of structural layers, the moisture content of the subgrade is regarded as of great importance in assessing the value of the modulus of deformation of the road pavement. This is also characterized by its own modulus of deformation, sharply changing during the year with variations of soil moisture content.

Shown *(Fig. 1)* are the variations of the modulus of deformation of silty-loam soil

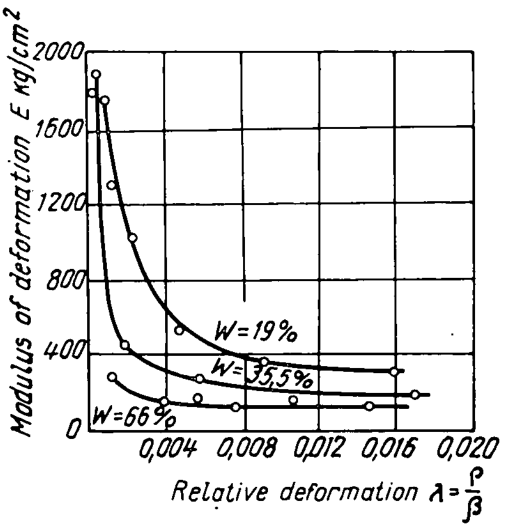

FIGURE 1

at different moisture contents calculated for several relative deflections.

3. The road pavement should be designed in such a manner that the absolute deflection (ρ) of the designed structure under the action of design load (p, B) does not exceed the allowable deflection taking into account the nature and intensity of road traffic.

The dual-tire wheel load of a design truck ($p = 5$ kg per sq cm and $B = 34$ cm) multiplied by the coefficient

$$K = 0.5 + 0.65 \lg N \qquad (2)$$

is taken as the design load in the USSR. The coefficient has been derived from laboratory and site observations. In formula (2) N is the number of design trucks passing along a two-lane road in 24 hours (traffic intensity) in the season of maximum, and simultaneously most frequent, weakening of the road pavement. This coefficient takes account of the influence both of repeated loads and the disturbance of the structure of the material and soil caused by the dynamic action of traveling vehicles.

The conversion of traveling automobiles of various types to the design automobile is performed by substituting a number of design automobiles of equivalent action on the road pavement for the actual number of traveling automobiles. As investigations have shown, the effect produced on the pavement by automobiles of various types may be determined with sufficient accuracy by the ratio between the products of unit pressure and the diameter of contact area.

The required modulus of deformation is to be determined by the following equation:

$$E_{req} = \frac{\pi}{2}\frac{p}{\lambda}(0.5 + 0.65 \lg N) \qquad (3)$$

where N, traffic intensity, is converted to the design automobile.

Usually for pavements with surfacing treated with organic binding materials, E_{req}, varies in the range of 400 to 700 kg per sq cm depending on the nature and intensity of traffic and also on the class of construction.

The method of design of flexible pavements employed in the USSR permits:

(a) The design of new road pavements with required operating properties (various values of λ and E_{req}) for various climatic conditions (E_o) with available data on traffic nature and intensity (N, p, B), and also data on the quality of material to be used (E_1, E_2 . . .).

(b) Varying designs of pavements of equal strength using various material in separate layers and creating, on the basis of comparison of these varying designs, the most expedient and economical structures.

(c) Estimates of the strength of existing pavements on existing roads, and on this basis the requirements for their improvement or restriction of traffic both as regards nature and intensity in the rainy period of the year.

The numerical values of deformability indices for pavements and subgrades (modulus of deformation) are obtained in the USSR as follows:

(a) By the analysis of data on the operation of existing road pavements.

(b) By testing of existing road pavements and subgrades using special mobile loading frames, applying the statistical analysis of conditions of entry of water to subgrades.

(c) By laboratory tests of soils and road-building materials.

The investigation of the behavior under repeated loads of subgrades is of primary importance as ultimately the behavior of road structures is determined by the condition of operation of the subgrade soil.

The results of tests by repeated loads applied through the bearing plate, and also by the traveling wheel on the experimental circular track, have shown that the regularity of accumulation of deformations and final condition of the soil, material or road structure after repeated loading, depends primarily on the relative value (in relation to the critical load, which causes failure at one application), of the acting repeated load.

In all cases the progressive accumulation of plastic deformations leading finally to failure was experienced at values of repeated loads lower than that of the destructive static loads. The degree of influence of repeated loads, however, was different for soils, materials and structures of different composition and moisture content. Thus, on moderately moistened clay and silt soil, road pavements with subgrades consisting of clay and silt soil, and also wet gravel-soil and crushed rock-soil mixtures progressively accumulate plastic deformations at repeated loads equal to 0.4-0.5 of the destructive static loads *(Fig. 2)*.

Wet clay and silt soil, as well as road pavements based on wet soil showed intensive accumulation of plastic deformations and final disintegration even under the action of considerably lower repeated loads (0.1-0.2 of the destructive static load) *(Fig. 3)*.

On the basis of the results of experiments performed in the USSR it is found that the design of high-duty and high-type road pavements with existing requirements as regards surface irregularities may be carried out by not allowing plastic displacement in the structural layers, i.e., considering that they must work in the elastic-viscous deformation stage.

The most widespread types of bases for flexible road pavements in the USSR are layers of rolled crushed rock, gravel or blast furnace slag placed on a stable subgrade. The stability of the subgrade is artificially increased by choice of high-quality soil, compaction of this soil, and elimination of the harmful influence of ground water.

Road structures sometimes contain layers of soil stabilized with mineral, cement or lime, and organic binding agents, bitumen and tar.

Sand and other similar porous layers, which are included in the road structure for drainage of the road pavement foundation are widely employed in regions with adverse moisture conditions of the road bed such as high ground water-level, sharp changes of temperature or deep and durable freezing, as a measure preventing frost heave. In these cases the porous layers are included in the calculations for finding the thickness of the road pavement and taking account of their modulus of deformation, together with the other structural layers.

The final choice of the base structure is made on the basis of technical and economical comparison of alternatives the equal strength of which is checked by compara-

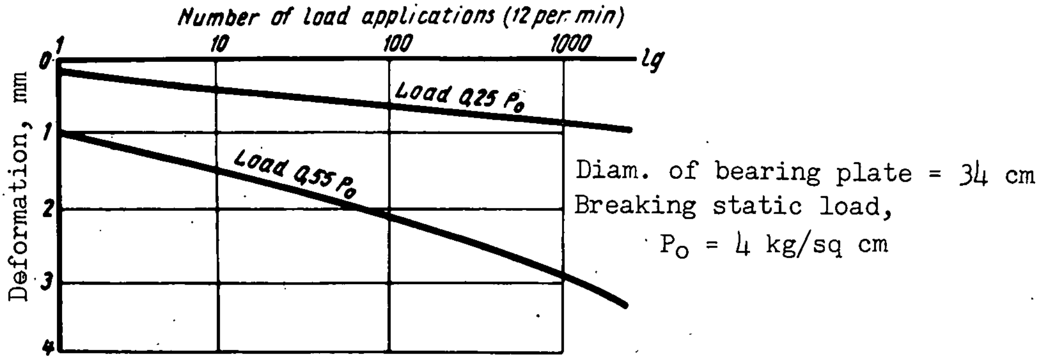

FIGURE 2

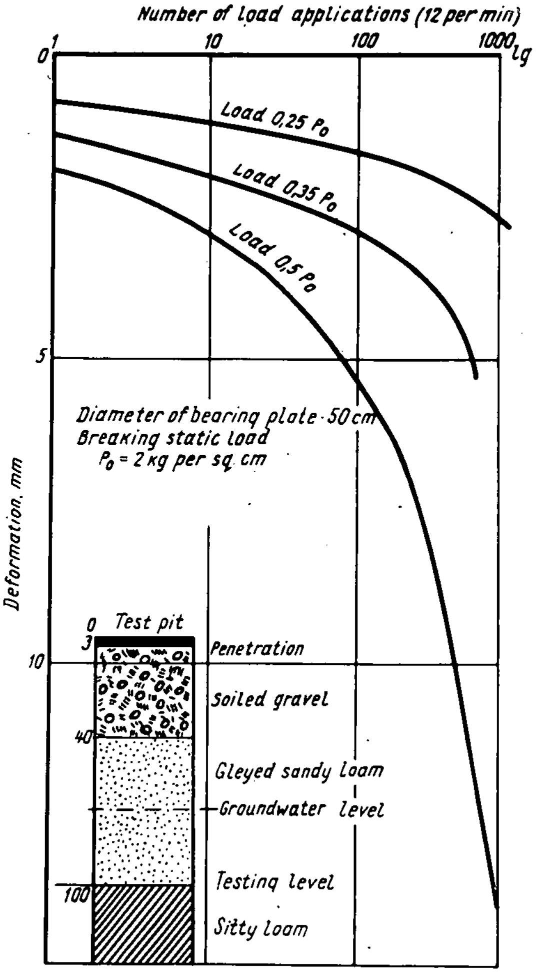

FIGURE 3

tive calculations according to the above mentioned method of maximum allowable deformations.

The initial data for designing flexible road pavement structures is the modulus of deformation of the subgrade, taking into account its stability during the year.

In the USSR it is considered that when new pavements are being designed the increase of their total strength should not be provided only by placing stronger layers in the upper part of the structure, because these layers are very expensive. It is more economical to develop the pavement in depth, taking measures to attain a stable value of the modulus of deformation of the subgrade until the depth is attained where the stresses of automobile loads, as well as the deformations caused by seasonal soil freezing, are not excessive.

The modulus of deformation of the subgrade should be related to the "design soil condition," characterizing the degree of its compaction and moisture content in the most unfavorable period of operation of the subgrade, taking into account the probability of development of this condition during the life of the road pavement. In the USSR the "design conditions," as the combination of weather conditions, moisture content and degree of soil compaction, are based partly on long-term observations of moisture condition of the road-bed and of the performance of road pavements at permanent experimental stations.

For maximum utilization of the bearing capacity of the soils and to reduce the thickness of the pavement structure, the degree of moisture of the road-bed soil by surface and ground water is limited by various structural measures. The principal measures are the provision of adequate drainage, ensuring a minimum elevation of the bottom of base layer above the ground water level and ground level, by building the road-bed on embankments with proper soil compaction, and by lowering the ground water level in cuttings by means of drains.

The choice of fill height, the thickness of frost-resistant layers and the specifying of the requirements for materials in the frost-resistant layers are dependent on climatic and hydrological conditions of the region and soil type.

On the basis of the reduction of stresses and the climatic effects by depth, it is best to arrange the structural layers of the base in such a manner that the degree of rigidity of successive layers decreases by depth in step with the reduction of compressive stresses while the frost-resistance decreases in step with the reduced temperature gradients.

This construction is considered the most economical, as not only high quality materials may be used, which are sometimes difficult to find at building sites, but also comparatively weak local materials, the use of which is not allowed as a rule, for layers at or near the surface. It is also considered advisable that the ratio of the moduli of deformation associated with the most unfavorable conditions of operation of adjacent layers of road structures be equal to at least 1.5 and not over 2.5-3.5, subject to the condition that the thickness of these layers will not be larger than the radius of wheel contact area B (2).

The criterion of the degree of soil compaction in the USSR is "standard compaction," which is similar to the well-known Proctor method. The "standard compaction" method differs from the Proctor method in that impact compaction of the soil sample is performed with a number of load impacts, determined for each soil when plotting the curve of relationship between the dry density of the soil and the number of compaction impacts. For "standard compaction" that number of impacts is chosen at which the dry density of the soil begins to approach asymptotically some constant value.

Properly performed soil compaction may substantially increase the subgrade resistance to loading and consequently create the conditions which admit of reduction of thickness of road pavements. Data is available showing that with thorough soil compaction the modulus of deformation of the soil may be increased by 40 to 50 per cent.

It is considered in the USSR that taking into account this restriction for the upper fill layers in the freezing-thawing zone, the degree of compaction should be not less than 95 per cent of the maximum value by the "standard compaction" method for non-cohesive silty soils and not less than 98 per cent for cohesive soils. The degree of compaction of those fill layers which are located below the freezing level, should be specified, taking into account the actual load and possible future moisture content of the soil.

In the USSR chemically stabilized soils are considered as high quality materials for building road foundations at places with poor resources of natural rock materials. Soil bases may be stabilized both by inorganic (cement and lime) and organic (bitumen, tar and synthetic resin) binding materials. Surface-active substances are also used, to widen the range of soils that may be stabilized and to lower the dependency of construction methods on seasonal conditions.

The principal soils stabilized in the USSR are clayey and silty soils. When considering stabilization these soils are regarded as complex colloidal-dispersion, polymineral and aggregated systems possessing sharp sorption capacity in relation to water, exchange ions and binding agents.

Special attention is being paid to the creation of optimal conditions for the formation of new, strong structural bonds by ensuring a high degree of soil pulverization and uniform mixing with the stabilizing reagent and thorough compaction during the operations for stabilization.

The investigation of the methods for efficient stabilization of soils with cement and lime and practical experience of the application of these methods for road bases constructed in the Soviet Union under the most variable soil conditions has shown that soils may be classified by the degree of their suitability for this type of stabilization into three groups.

Soils which do not meet the necessary requirements are not fit for stabilization with cement and lime in their natural condition, and the possibility of their utilization is determined by special laboratory tests.

It is not advisable to stabilize with cement soils having an acid reaction, pH below 5, as the acid media of the mortar filling the soil pores retards hydration and hydrolysis of the cement grains, due to which, even at cement contents up to 15-20 per cent, the soil does not become sufficiently stable and water resistant.

When carbonate soils conforming to the requirements (for example, loess, loess-like loam or grey desert soil) are stabilized with portland cement, the processes of hydrolysis and hydration proceed, on the contrary, in the most intensive way and the attained strength of the soil-cement is reached, other conditions being equal.

An important factor in determining the strength and water resistance of soil-cement is the quantity and activity of the cement. The best results in soil stabilization are obtained when adding portland cement with an activity of not less than 400 kg per sq cm.

If frost and water do not act together over a long period it is also possible to use lime for subgrade soil stabilization. In the latter case highly active lime, containing not less than 70 per cent of CaO, both slaked and unslaked, in ground form may be used. The use of lime more than 30 days old is prohibited.

Medium and quick setting cut-back bitumen and coal tar are mostly used in the USSR for soil stabilization. These materials in favorable climatic, geological and hydrological conditions, make sufficiently strong and stable mixtures. The compressive strength of soil stabilized with an optimum (7-11 per cent) amount of bitumen or tar in dry and warm climatic conditions varied in the range of 10 to 15 kg per sq cm; the modulus of deformation being 600 to 700 kg per sq cm.

Saline and salinous soils may be stabilized with bitumen and tars only when first heated with a solution of calcium chloride;

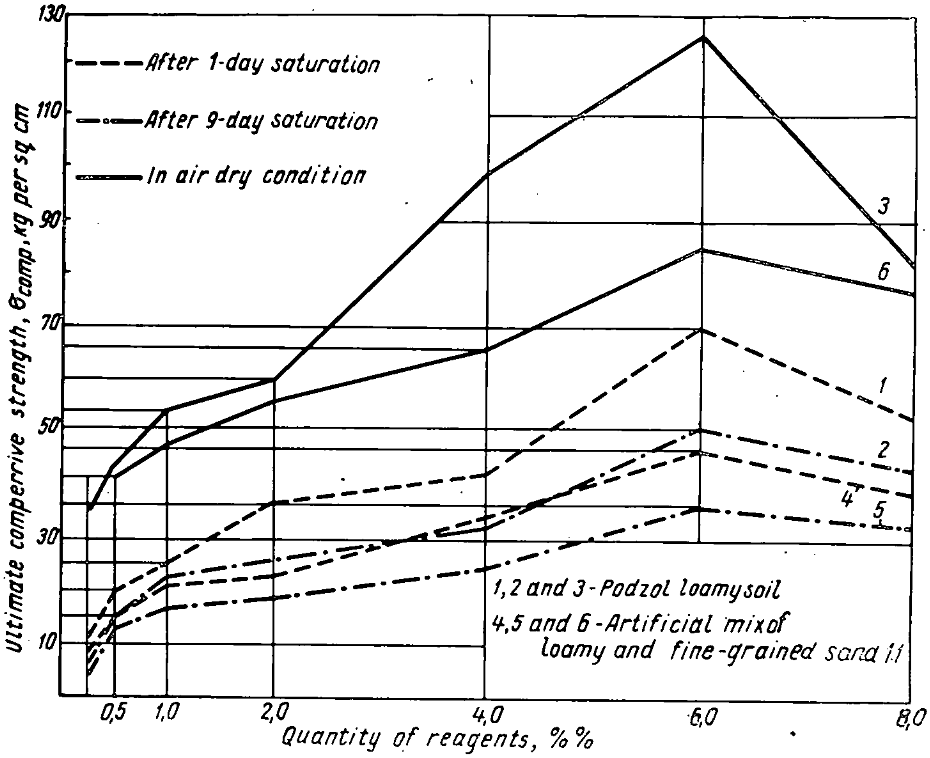

FIGURE 4

this changes salt conditions of the soil and ensures a water content near to the optimum moisture content. To increase adhesion as well, sodium salts of organic acids may be added to the bitumen.

Available experience shows that for stabilization with cut-back bitumen and tar various types of sandy loam, silty and clayey loam soils with a plasticity index below 17, may successfully be used in the central zone of the European part of the USSR. Carbonate varieties of soils and humus soils are used with a plasticity index up to 25. Soils of excessive plasticity are improved with bitumen or tar only after adding leaning additives of sand or sand-gravel material.

Soil stabilization with cut-back bitumen and coal tar in regions with excessive precipitation and comparatively cold climatic conditions is not as efficient and besides there are also technological difficulties, caused by excessive soil moisture content. In these conditions only sandy loam and light loamy soils with a plasticity index not over 12 are efficiently stabilized with bitumen.

Furforal aniline, naphtha soap and other surface-active reagents are also used for soil stabilization. When these materials are being considered, it is necessary to take into account the mineralogical constitution of the clayey part of the soil to ensure maximum ionic sorption, which lowers the surface energy of the soil and develops forces of ionic bond. An example may be the hydrophobic strengthening of non-carbonate acid loamy soil by its treatment with small quantities of furfurol aniline (1-2 per cent). Soils stabilized with synthetic furfurol aniline resins may be used successfully as bases of road pavements, as water-insulation, and as stronger layers.

When the amount of the reagent is increased, the rise of strength is regular up to a certain optimum value after which the soil strength drops, this being connected with the cohesion of the reagent, which is frequently less than its adhesion. In saturated condition the variation of strength of hydrophobized soil is of the same character, but with somewhat lower ultimate strength values *(Fig. 4)*.

Stabilization of Settling and Weak Clayey Soils by Thermal Treatment

I. M. Litvinov

Settling loess soils are very abundant in many countries and especially in the Soviet Union where they cover vast areas. Numerous cases of excessive differential settlement occur on these soils, often followed by collapse of various buildings and other structures owing to the high compressibility of such soils when wetted under applied load.

This has already brought about great damage and will do so in the future, for an immense number of large buildings and structures have already been erected on settling soils and the rate of important construction on such soils is steadily increasing.

Different methods of loess soil stabilization have been suggested by a number of investigators. These methods, however, do not ensure the degree of consolidation required, or otherwise involve too much cost and labor.

The Southern Research Institute for Industrial Construction (Academy of Construction and Architecture, Ukrainian SSR) has developed different methods for the thermal consolidation of loess soils *(Fig. 2)*.

Thermal treatment of loess and other soils can be accomplished by two methods.

The first method, attributed to N. A. Ostashev, consists in blowing hot air under pressure into the soil through heat-proof pipes and bore holes, the air having been heated to a temperature of 600-800° C in special stationary or movable furnaces. This method has been found not to be sufficiently effective, hence it is not used in construction work.

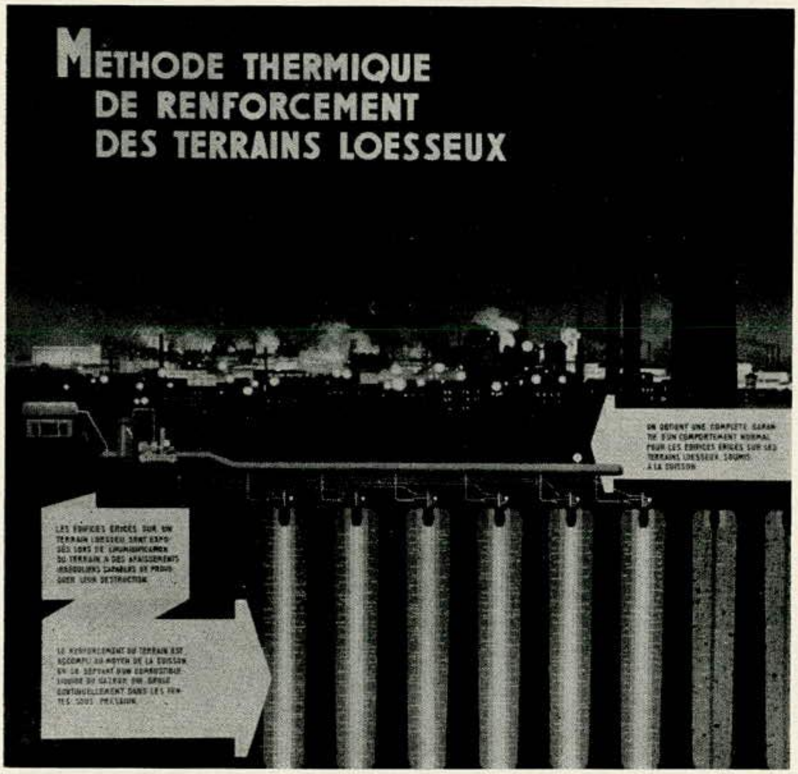

FIGURE 1
Thermal consolidation of soils. Photograph taken for the Brussels International Exhibition, 1958.

The second method, as offered by the author, has found wide application on building sites. It involves burning various fuels in the soil being treated, the process of combustion taking place in sealed bore holes with control of the temperature and chemical composition of the combustion products *(Fig. 3)*.

Heating of the soil to a temperature high enough to cause the necessary changes in the soil characteristics is achieved mainly by infiltration of the compressed heated air or of the incandescent products of combustion through the pores in the soil.

This method, which has been successfully applied in practical construction work, involves less complicated equipment and less labor while being more effective and economical than the first method, all of which facilitates and extends the range of its application.

In this paper the author describes the basic requirements which must be met in using the second method of thermal stabilization *(Fig. 4)* applicable to settling loess and other soils of porous structure.

By using the thermal method of consolidation the settling properties of loess soils can be entirely eliminated to a depth of 10 to 15 meters below the footing base, while the load bearing capacity of these soils is greatly increased *(Fig. 5)*.

From the engineering and economical standpoint deep thermal treatment of soils should be recommended:

(a) To consolidate loess soils in the foundations of important residential and industrial buildings, as well as other engineering and special types of structures to be erected, which do not allow differential settlement.

(b) To eliminate the possibility of failures of various existing buildings and structures due to excessive differential settlement.

(c) To prevent landslides and many other causes of failures.

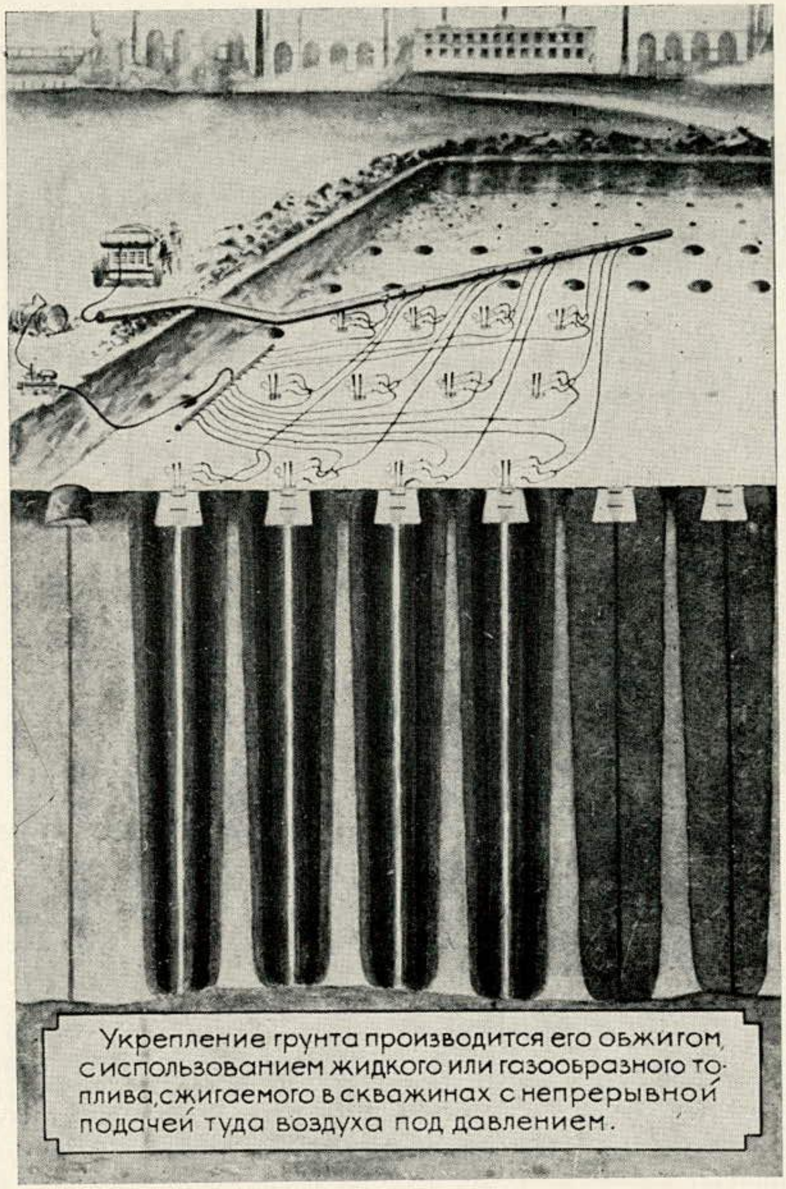

FIGURE 2
General scheme of the procedure for thermal treatment of soils.

Apart from thermal processing, the second method also enables thermo-chemical treatment of soils by means of the hot gaseous products of combustion to which special chemicals are added, if necessary. As a result of the joint effect of the incandescent gases and chemicals introduced before, during, and after thermal processing, thermal, thermo-chemical or combined consolidation of various soils is made possible *(Fig. 6)*.

Due to simple temperature control during soil firing (by blowing in different amounts of air per 1 kg of liquid or solid fuel or per 1 m^3 of gaseous fuel) in a wide range of temperatures (up to 200° C), the second method can be adopted not only for a uniform consolidation of large volumes of loess soil at temperatures between 300° and 1,000° C, but for other structural purposes

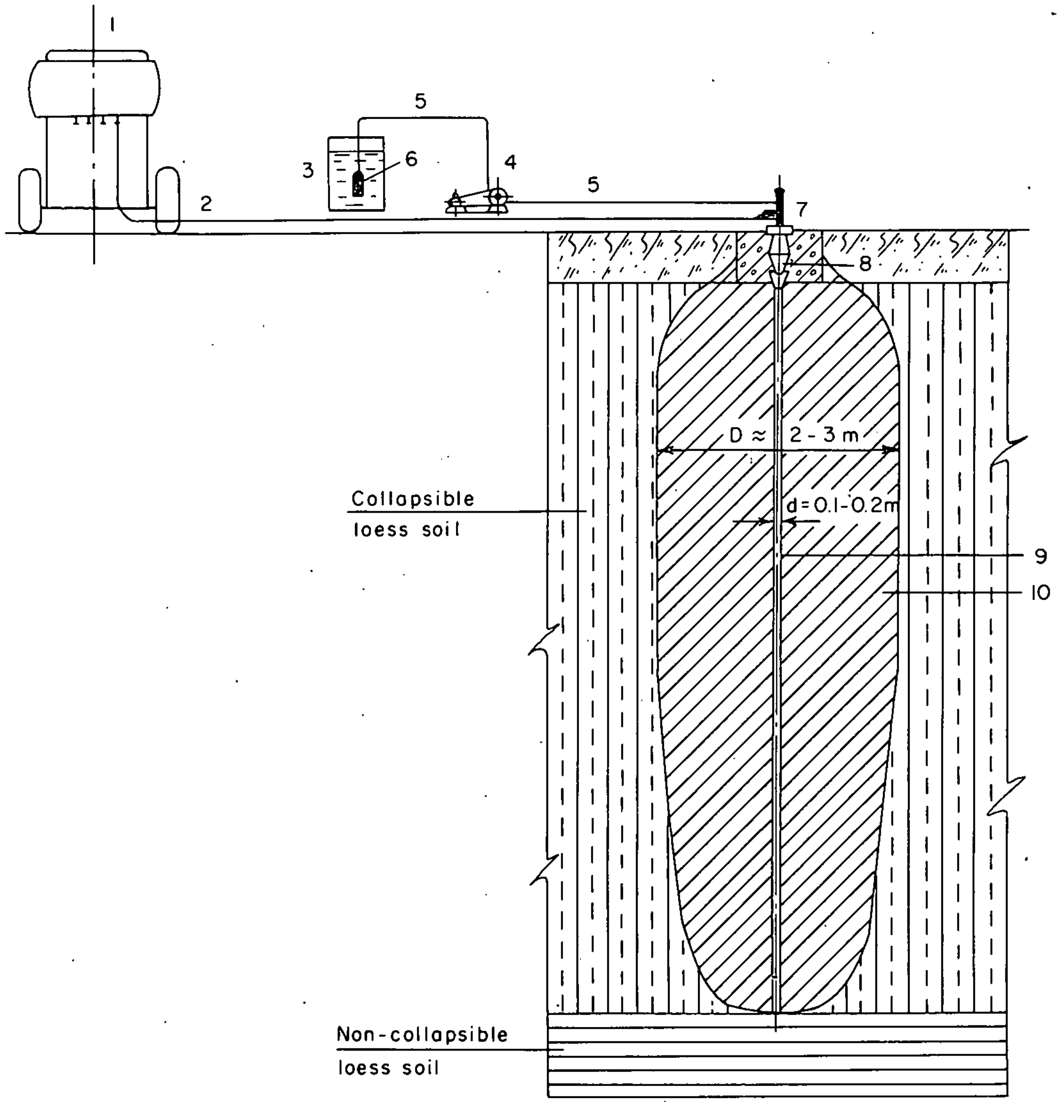

FIGURE 3

Diagram of installation for thermic stabilization of collapsible loess soils by the second method. 1. Compressor; 2. pipeline for cold air; 3. container for liquid fuel; 4. pump for supplying fuel under pressure into the bore hole; 5. fuel pipe line; 6. filters; 7. nozzle; 8. cover with combustion chamber; 9. bore hole; 10. zone of thermic stabilization of soil.

as well, when higher temperatures are required causing melting of the ground.

To facilitate the penetration of the incandescent air into the soil, the pressure of the hot gases should be maintained above that of the atmosphere by pumping cold air into the bore holes. Raising the excess pressure greatly increases the effectiveness of thermal treatment and improves the technical and economical characteristics.

The temperature of the products of combustion must not exceed fusion temperature of the soil to be consolidated; this is easily ensured by regulating the amount of cold air supplied. The amount of fuel required per unit of time (kg per hour for solid and liquid fuel or m^3 per hour for gaseous fuel) is determined in accordance with the permeability of the soil.

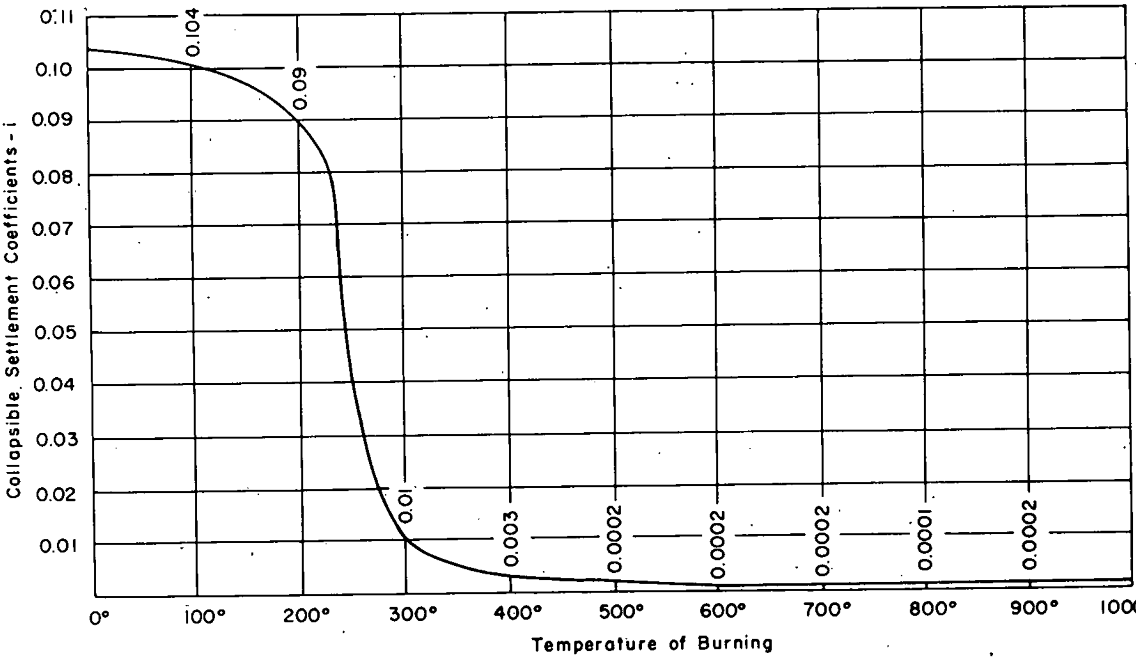

FIGURE 4
Variation of the collapsible settlement coefficients of loess soils as a function of the temperature of burning.

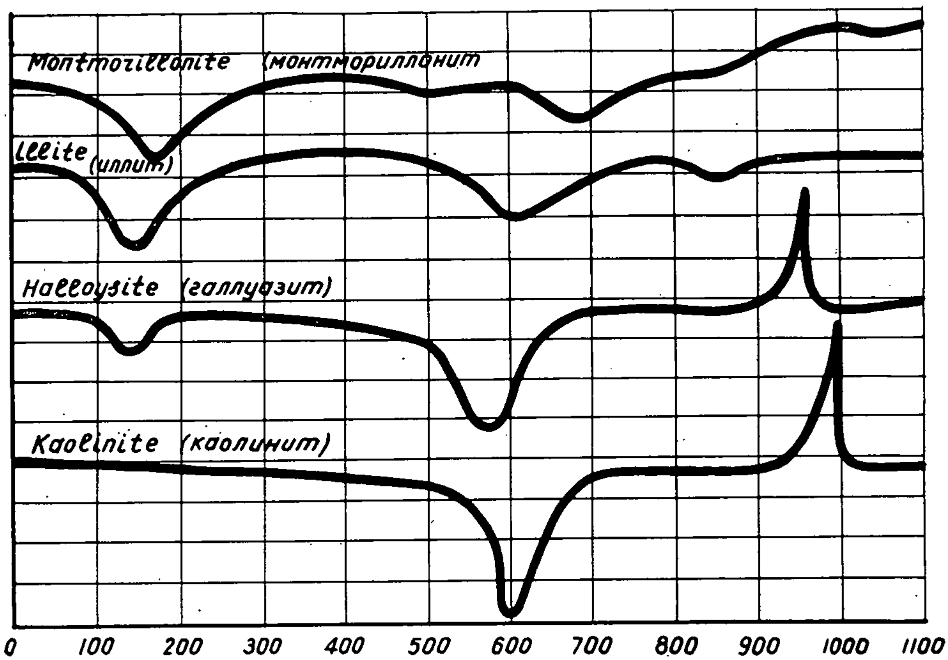

FIGURE 5
Differential thermal diagrams for the four basic constituents of clay.

No.	Characteristics of Soil Samples	Coefficient of Collapsible Settlement under $p=3$ kg/cm²	Compression Modulus under $p=3$ kg/cm²		Magnitude of Shear Load in kg/cm² under: $p=$ 1 kg/cm²		$p=$ 2 kg/cm²		$p=$ 3 kg/cm²		Angle of Internal Friction ϕ (°)		Coeff. of Internal Friction f		Cohesion c kg/cm²	
			Dry	Sat.	Dry	Sat.	Dry	Sat.	Dry	Sat.	Dry	Sat.	Dry	Sat.	Dry	Sat.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
	City of Zaporozhye, Block No. 84 (1957)															
1	Before burning	0.1030	56.6	148.4	0.60	0.36	0.98	0.70	1.60	1.20	21	19	0.38	0.34	0.22	0.22
	At 300 to 500 C	0.0067	12.9	13.5	3.80	2.40	—	—	5.50	4.00	40	39	0.85	0.80	2.95	1.47
	At 500 to 700 C	0.0005	10.5	11.2	—	—	—	—	—	—	—	—	—	—	—	—
	At 700 to 900 C	0.0002	9.6	9.5	4.3	2.50	5.30	—	5.90	4·50	40	44	0.85	0.98	3.50	1.60
	City of Dniepropetrovsk, Firehouse Depot (1957)															
2	Before burning	0.0650	90.0	160.0	0.80	0.30	1.20	0.70	1.80	1.00	27	19	0.50	0.35	0.30	0.05
	At 300 to 500 C	0.0003	24.0	24.2	2.50	2.30	3.30	3.00	4.20	3.70	40	35	0.85	0.70	1.65	1.60
	At 500 to 700 C	0.0002	13.5	13.6	—	—	—	—	—	—	—	—	—	—	—	—
	At 700 to 900 C	0.0000	12.8	12.8	5.00	4.80	5.80	5.60	6.80	6.60	42	42	0.90	0.90	4.10	3.90
	City of Bagley, Coke Chemical Plant (1957)															
3	Before burning	0.0500	90.0	162.5	0.70	0.40	1.10	0.70	1.60	1.00	24	17	0.45	0.30	0.25	0.10
	At 300 to 500 C	0.0008	23.0	23.8	2.40	2.20	3.30	2.90	4.10	3.70	40	37	0.85	0.75	1.55	1.45
	At 500 to 700 C	0.0001	12.5	12.6	—	—	—	—	—	—	—	—	—	—	—	—
	At 700 to 900 C	0.0000	12.0	12.0	4.50	4.00	5.70	5.30	7.00	6.30	51	49	1.25	1.15	3.25	2.85
	City of Nikopol, Southern Pipe Plant (1958)															
4	Before burning	0.0590	90.2	149.2	—	—	—	—	—	—	—	—	—	—	—	—
	At 300 C	0.0175	30.2	47.7	—	—	—	—	—	—	—	—	—	—	—	—
	At 600 C	0.0044	24.0	28.4	—	—	—	—	—	—	—	—	—	—	—	—
	At 900 C	0.0008	15.1	15.9	—	—	—	—	—	—	—	—	—	—	—	—
	City of Upper-Dneprovsk, Starch Plant (1958)															
5	Before burning	0.1340	26.0	160.0	—	—	—	—	—	—	—	—	—	—	—	—
	At 300 C	0.0227	12.1	34.8	—	—	—	—	—	—	—	—	—	—	—	—
	At 600 C	0.0060	10.4	16.3	—	—	—	—	—	—	—	—	—	—	—	—
	At 900 C	0.0027	9.2	11.3	—	—	—	—	—	—	—	—	—	—	—	—

Note: The color of the soil before burning was pale yellow; at 300 C, yellow-pink; at 300 to 500 and 600 C, pink, at 700 to 900, red.

FIGURE 6

Variation of the most indicative physico-mechanical properties of collapsible loess soils under the action thereon of different temperatures.

In this second method, the use of gaseous fuel is especially effective, since starting the firing of the bore holes is greatly simplified, the walls of the bore hole, and thus the soil layers, are more evenly heated, temperature control in the bore hole is improved, better conditions are created to prevent partial fusion of the walls of the bore hole, which is not permissible, and the total cost of thermal treatment is considerably lowered. The cost of gaseous fuel, as shown by information from the site, is but a small part (about 3 per cent) of the total cost of the thermal treatment of soil, about 70 per cent of the total cost involving air and boring expenses *(Figs. 7, 8)*.

The burning of the fuel, gaseous, liquid or solid, is done in the mouth of the bore hole or directly in the soil mass itself. The mouth of the bore holes above are tightly closed by special shutters and an excess pressure of the hot gases, 0.25 to 0.50 atm, is permanently kept above atmospheric pressure. There being no outlet, the incandescent gaseous products of combustion infiltrate through the pores in the ground and heat the soil mass to the temperature required.

If air sources of sufficient power are available ensuring an excess pressure of 0.25 to 0.50 atm, the thermal treatment of large volumes of ground can be carried out simultaneously.

The heat transfer from the hot gases in the bore hole to the soil mass is achieved in

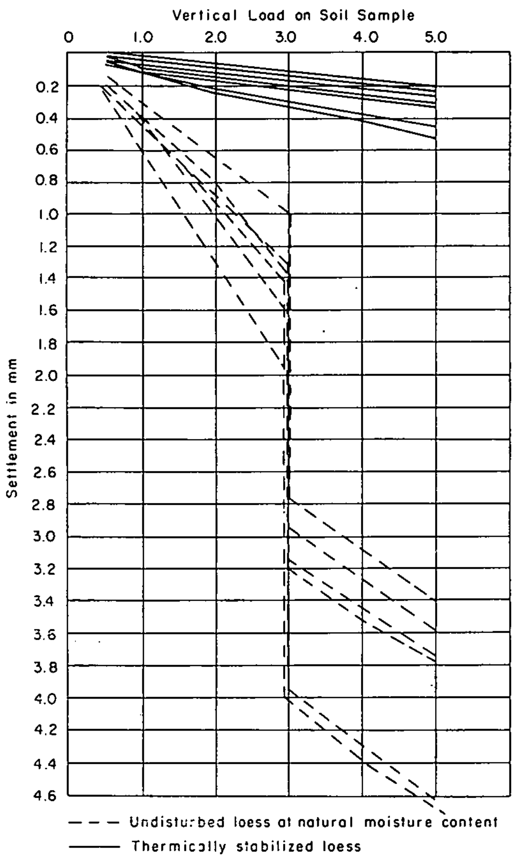

FIGURE 7

Composite results of compression tests of loess samples before and after thermal treatment.

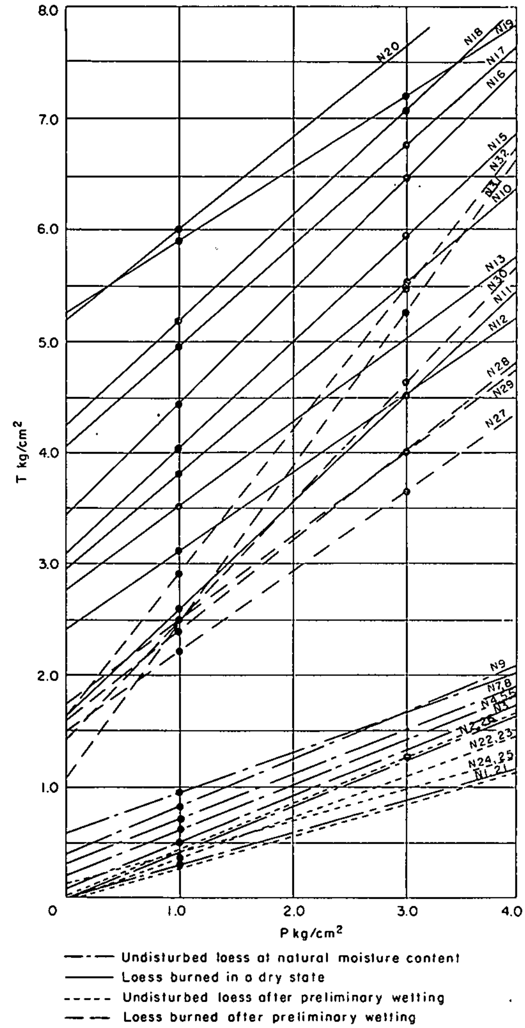

FIGURE 8

Composite results of shear tests of undisturbed loess (at natural moisture content and after preliminary wetting) and of thermically stabilized loess (dry and after preliminary wetting).

two ways: mainly by filtration of the air and incandescent gaseous products of combustion through the pores of the soil to be consolidated, and by direct transmission of heat due to temperature difference and the contact between the heat source and the surface of the soil.

Settling loess soils when subjected to thermal treatment greatly change their physico-mechanical properties, viz.:

(a) Ability to settle and to be wetted are entirely eliminated.

(b) Cohesion, compressive and shear strengths are greatly increased.

(c) Settlement under an applied load when the ground is wet immediately ceases.

(d) Color changes from natural pale yellow to various shades of red.

The temperature of the hot gases which are formed in the bore hole due to the combustion of the fuel can be controlled by changing the amount of air blown into the bore hole. By increasing or decreasing this amount of air the temperature of the combustion gases is raised or lowered, respectively. The excess air blown into the bore hole does not participate in the chemical reaction of combustion but merely mixes with the products of combustion and lowers the temperature of the mixture, serving as an additional heat carrier transferring the heat through the pores of the ground.

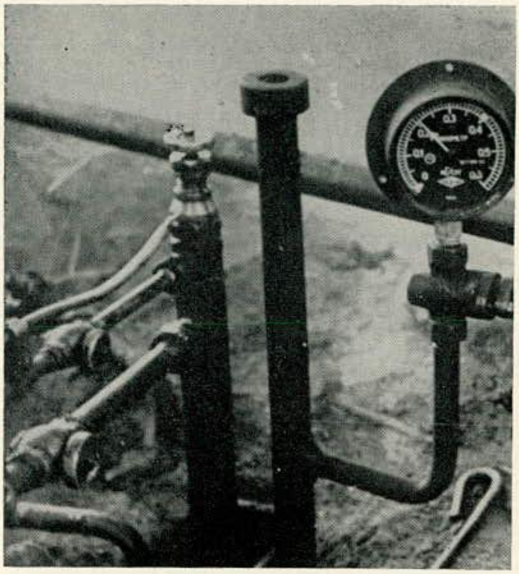

FIGURE 9
General view of the upper part of the bore hole being fired.

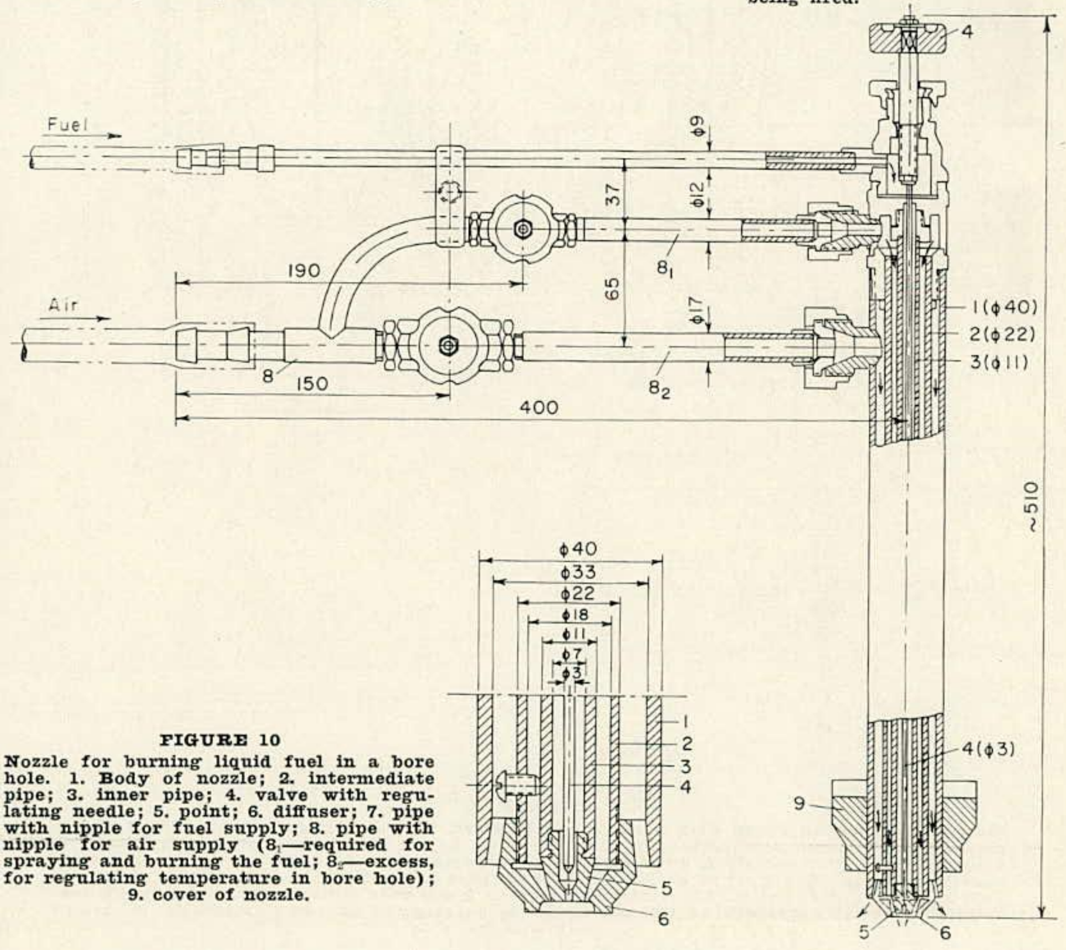

FIGURE 10
Nozzle for burning liquid fuel in a bore hole. 1. Body of nozzle; 2. intermediate pipe; 3. inner pipe; 4. valve with regulating needle; 5. point; 6. diffuser; 7. pipe with nipple for fuel supply; 8. pipe with nipple for air supply (8_1—required for spraying and burning the fuel; 8_2—excess, for regulating temperature in bore hole); 9. cover of nozzle.

The temperature of the gases in the bore hole (losses not taken into account) can be determined from the following equation:

$$t_r = \frac{Q_r}{(1.293\ V_B + 1) \cdot c_p}$$

in which

Q_r = the calorific value of the fuel;

V_B = the amount of air blown into the bore hole per 1 kg of fuel (m^3);

c_p = the average heat capacity of the products of combustion at constant pressure p (kg.-cal. per kg per deg) which is taken equal to 0.235 + 0.000019) t_r.

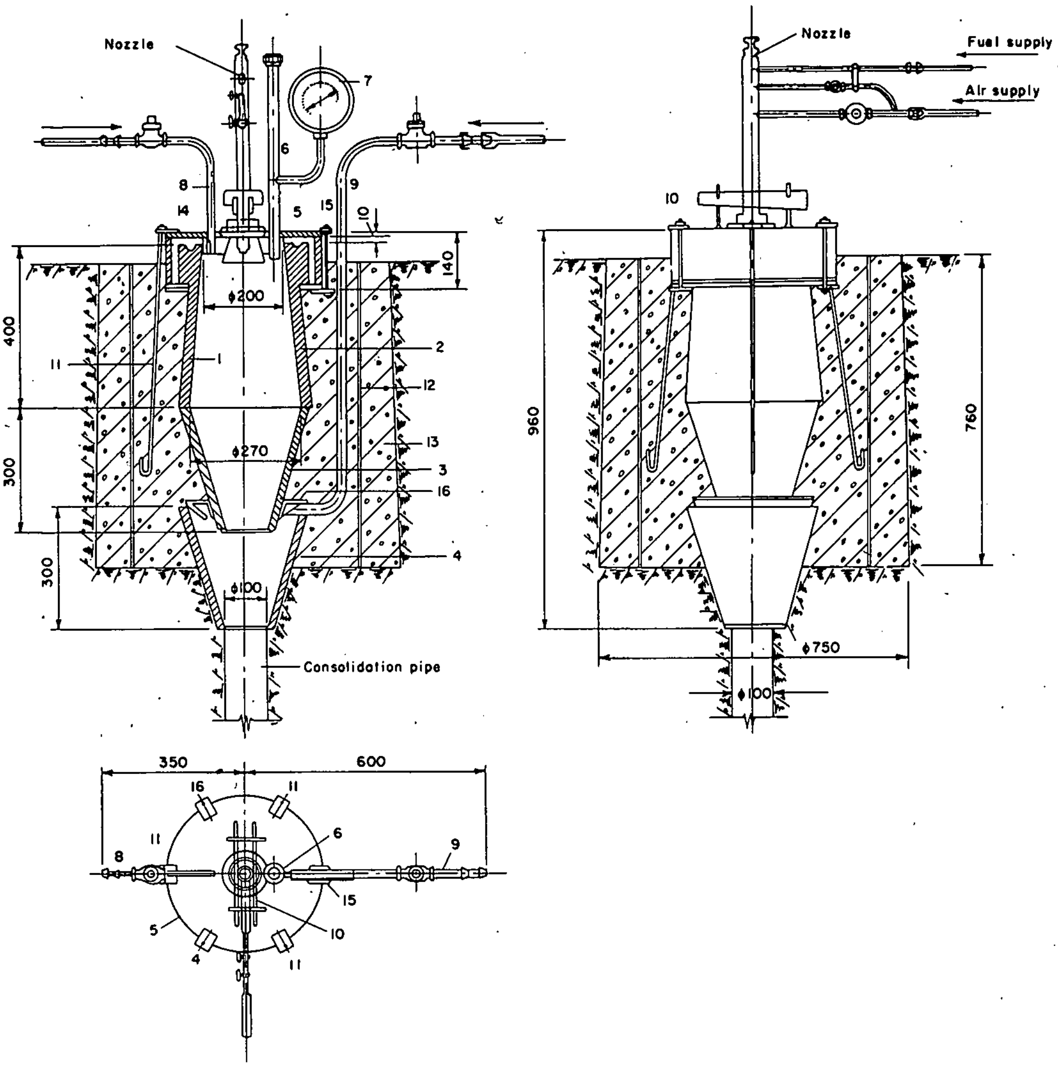

FIGURE 11

Hermetically sealed cover with combustion chamber. 1. Combustion chamber; 2, 3 and 4. ceramic cones lining combustion chamber; 5. metal lid; 6. observation pipe, with branch to manometer; 7. manometer; 8. pipe for supply of excess air to upper part of combustion chamber; 9. pipe for supply of excess air to lower part of combustion chamber; 10. wedges to hold nozzle; 11. anchor ties; 12. reinforcing steel of 6-mm rods (total weight 3-4 kg); 13. concrete of red brick aggregate; 14. thermo-insulating packing; 15. fixation of metal lid; 16. hollow ring with lye.

In Table 1 is given the approximate theoretical relationship between the amount of air blown into the bore hole per one kg of fuel and the temperature of the gases in the bore hole for liquid fuel (Diesel fuel):

TABLE 1

$\frac{V_B \text{ m}^3/\text{kg}}{V_o \text{ m}^3/\text{kg}}$	1	1.5	2	2.5	3	3.5
V_B m³/kg	11.2	16.8	22.4	28	33.6	39.2
t_r degrees	2800°	1670°	1300°	1050°	896°	785°

The amount of air that is blown inside the bore hole (V_B) should be 2.5 to three times the minimum quantity which is required for complete fuel combustion.

The amount of air filtering through the walls of the bore hole into the ground is dependent on the gas permeability of the soil and on the pressure in the bore hole and should be determined experimentally by a blowing test. With loess soils of 8 to 20 per cent moisture content the quantity of air that is filtering into the ground is usually

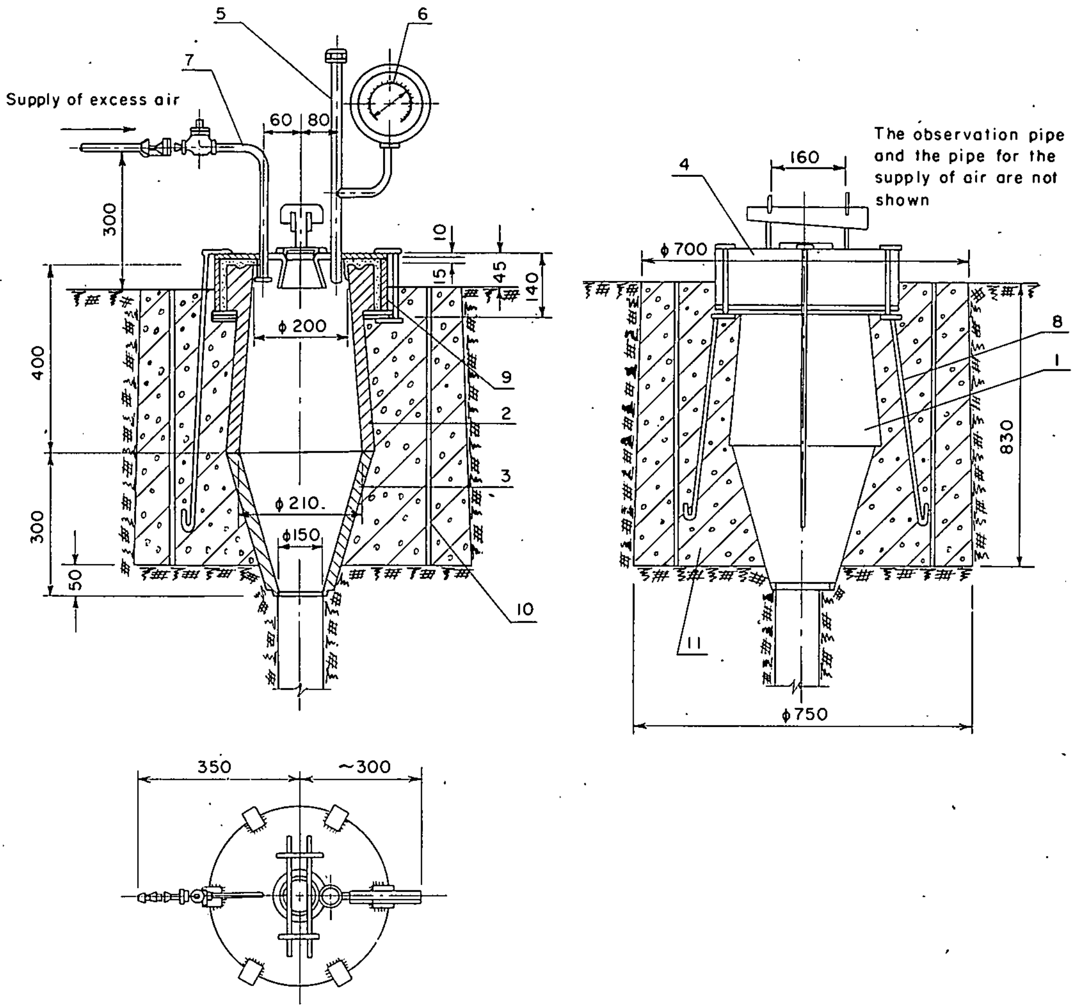

FIGURE 12

Hermetically sealed cover with combustion chamber for gas fuel. 1. Body of combustion chamber; 2 and 3. ceramic cones lining combustion chamber; 4. lid of body with wedge lock; 5. observation pipe with branch for manometer; 6. manometer; 7. pipe for supply of excess air; 8. anchor ties; 9. thermo-insulating packing; 10. steel reinforcing rods; 11. concrete of brick aggregates.

10 to 40 m^3/hr per 1 m bore hole depth *(Figs. 13, 14, 15)*.

The air quantity V_B that is necessary to provide for an optimum thermal treatment under conditions of complete fuel combustion and cooling of the combustion products (in m^3 per 1 kg of liquid fuel or 1 m^3 of gaseous fuel) is dependent on the temperature of the hot gases in the bore hole as determined from the formula above or from Table 1.

The quantity of fuel burned during one hour per 1 m run of the hole is determined from the caloric value of the fuel, gas per-

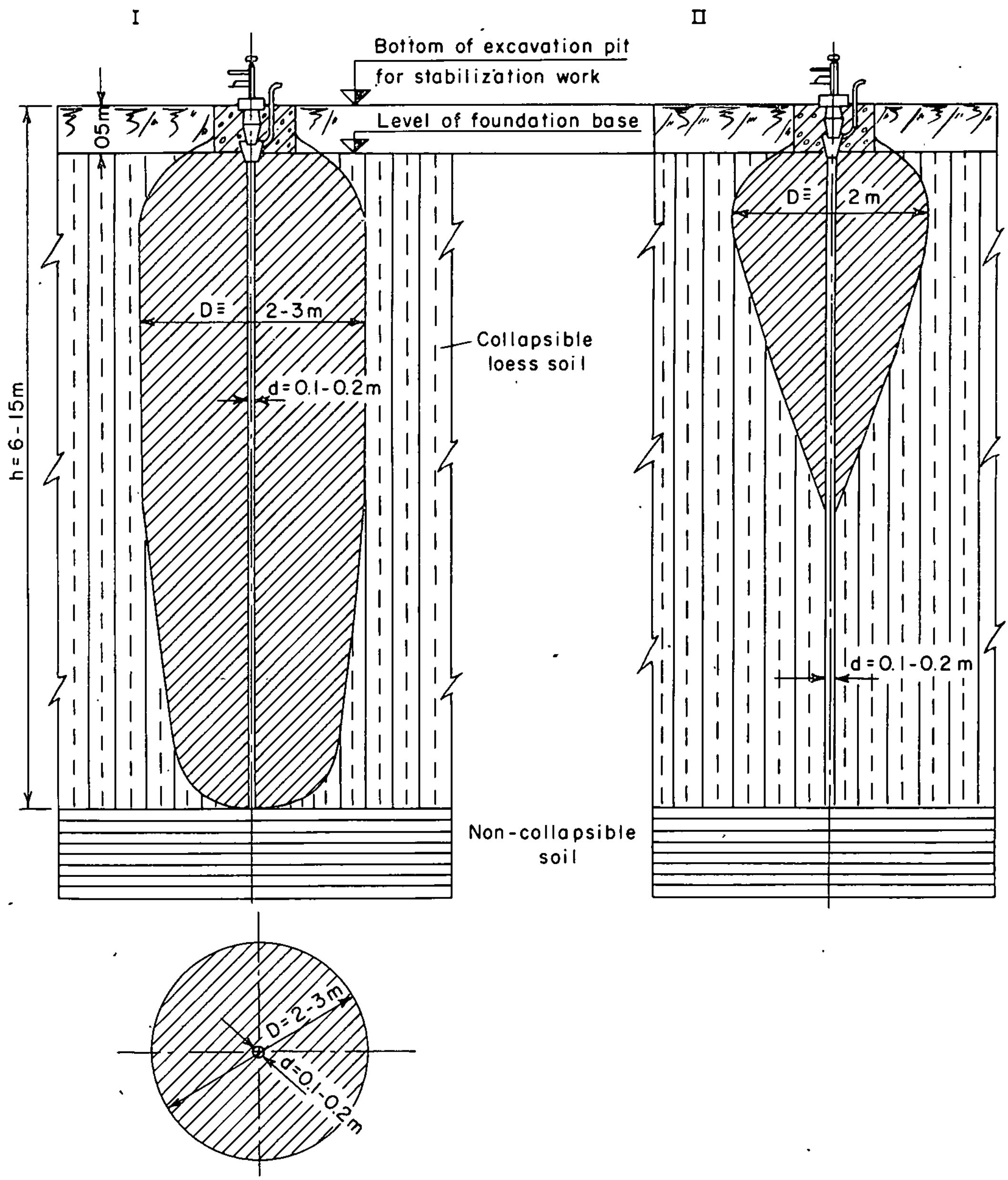

FIGURE 13
The spread of zones of thermic stabilization (strengthening) of soil around vertical bore holes. I. Under an excess pressure of 0.2-0.5 atm. in the bore hole. II. With no excess pressure in the bore hole.

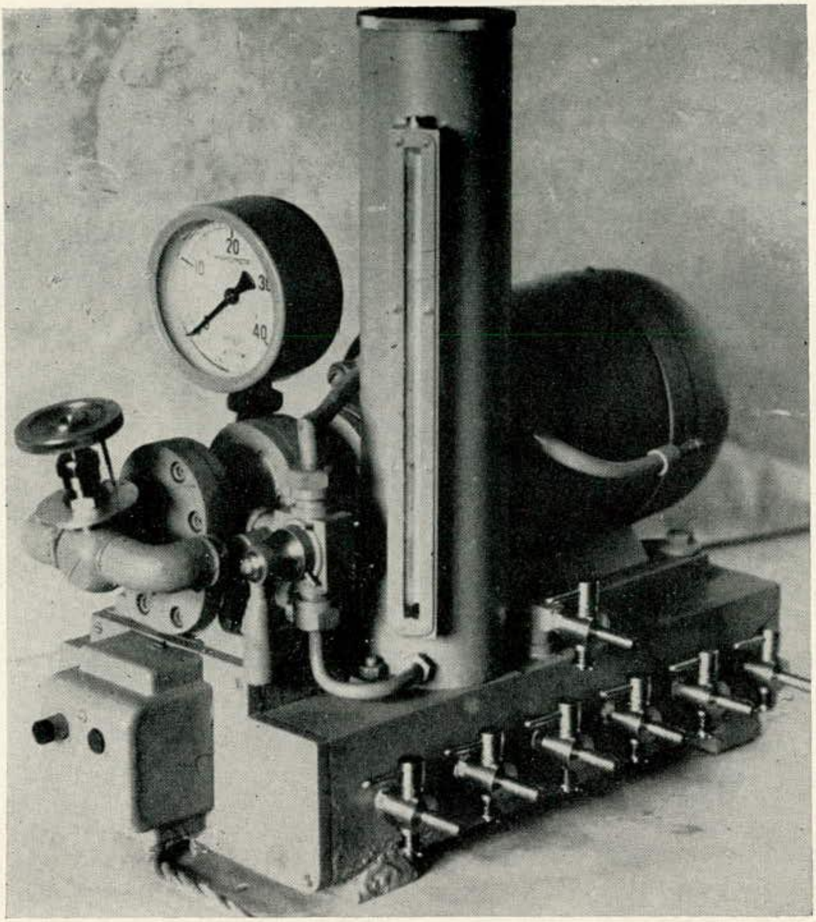

FIGURE 14
General view of equipment for pumping liquid fuel into 14 or more simultaneously fired bore holes.

meability, fusion temperature, moisture content and volume weight of the soil to be consolidated.

Increasing the quantity of fuel burned per unit time (1 hr) will raise the temperature of the hot gases above the calculated value and melting of the walls of the hole may take place; however, such a hole should be rejected and a new one drilled nearby.

The thermal treatment in one 15 to 20 cm dia. bore hole during a period of eight to 10 days will result in the formation of a consolidated zone of 1.5 to 2.5 m diameter and 8 to 10 m deep.

If the time duration of thermal treatment is increased, the consolidated zone around each hole will become larger (3 m diameter and 15 m depth or more, i.e., a volume of 100 m^3 and more of consolidated ground for one hole).

The thermal consolidation of soils is designed in one or several cycles with simultaneous thermal treatment of a corresponding number of bore holes in each cycle.

To increase the rate of processing the number of cycles should be as low as possible. Thus, in underpinning the foundation of various buildings or eliminating the consequences of failures due to local wetting of the soil, as well as in many other cases when the total number of fire bore holes varies from 6 to 30, the work of thermal treatment should be carried out in one or two cycles, i.e., during a period of 10 to 20 days *(Fig. 16)*.

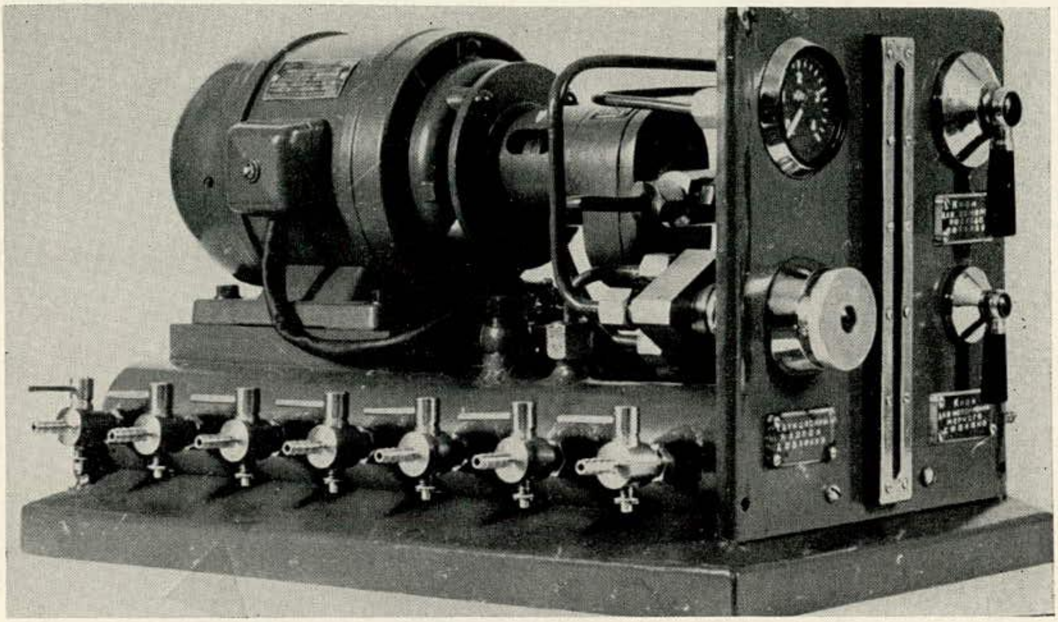

FIGURE 15
View of improved equipment with protective casing and automatic pressure regulation for pumping fuel into 15 to 30 simultaneously fired bore holes.

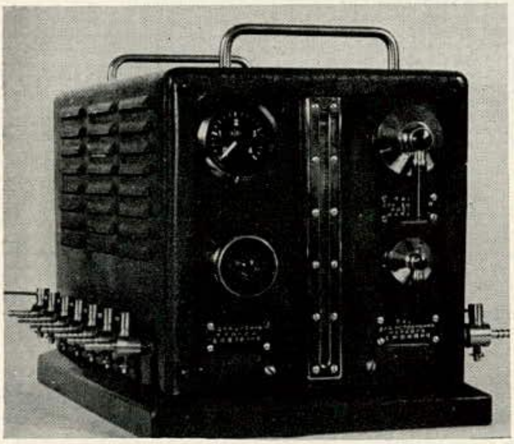

FIGURE 16
Simplified equipment for pumping fuel into six simultaneously fired bore holes.

The duration of each cycle of burning the calculated amount of fuel under the conditions assumed is about 10 days and may increase or decrease depending on the depth of the bore hole, the diameter of the consolidated zone and the output of the equipment for pumping air into the bore hole.

In applying the thermal method of consolidation, its economical and engineering advantages in the case under consideration should be taken into account. The application of this method is economically unjustifiable for underpinning the foundations of small and unimportant buildings and structures and when the thickness of the settling layer of soil is small *(Fig. 21)*.

In the course of thermal processing continuous control of the combustion process in the bore hole should be carried on by maintaining a temperature between 750 and 1000° C at pressures of 0.25 to 0.50 atm. The burning of the fuel can be observed through a special peep hole in the shutter.

The thermal treatment is considered complete when a calculated amount of fuel has been burnt inside the bore hole at a pressure not below 0.25 to 0.50 atm and when a proper amount of air has been pumped into the hole.

After firing has been completed and the ground inspected the bore holes are filled with soil and thoroughly rammed.

By applying the thermal method of consolidation a large number of damaged buildings and other structures have been saved from collapse and many have been

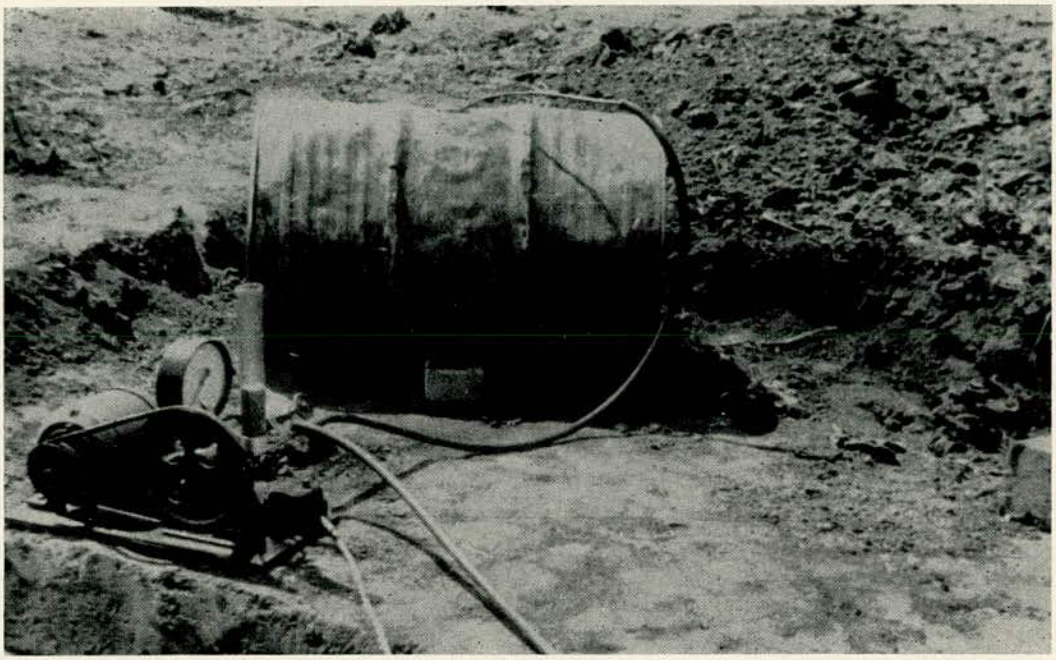

FIGURE 17
View of site during firing of bore holes by gaseous fuel (coke gas) at the Bagley Coke Chemical plant.

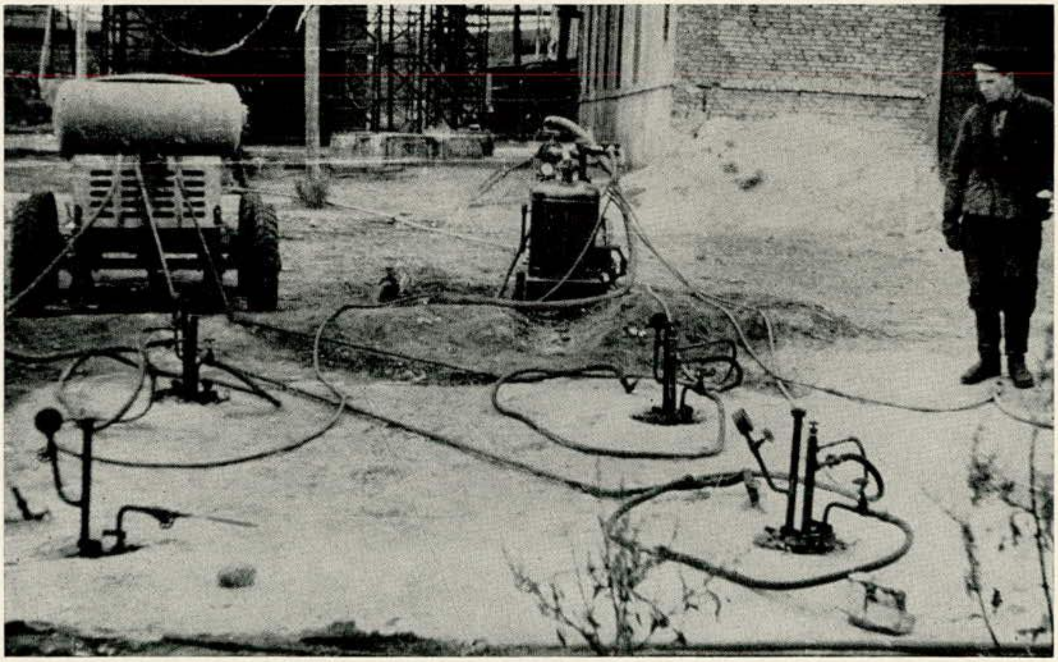

FIGURE 18
General view of site during thermal treatment of soil in a city block in Zaporozhye.

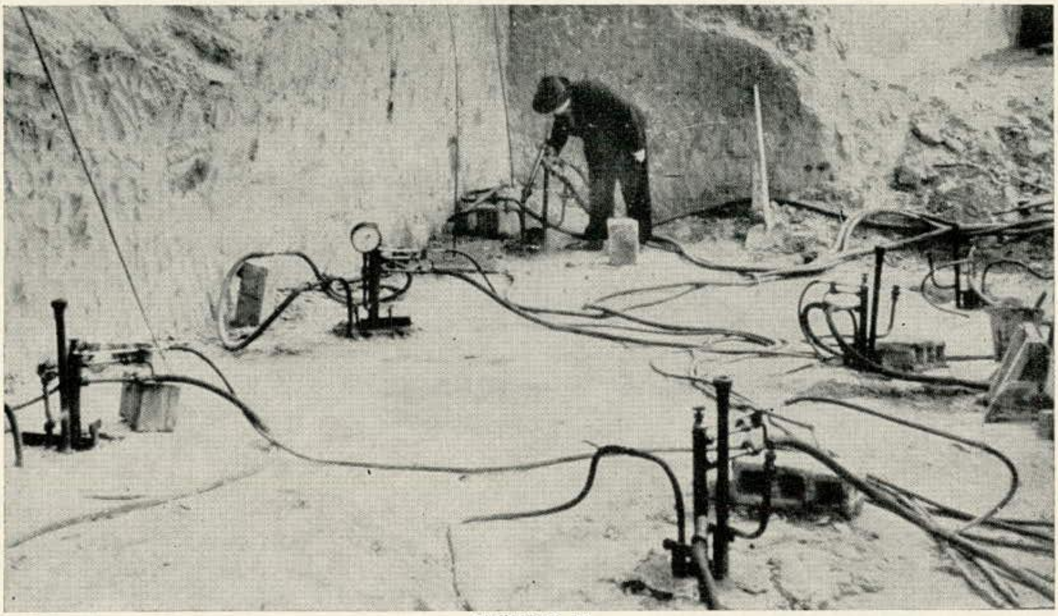

FIGURE 19
Example of the operation of equipment for 12 bore holes fired by liquid fuel.

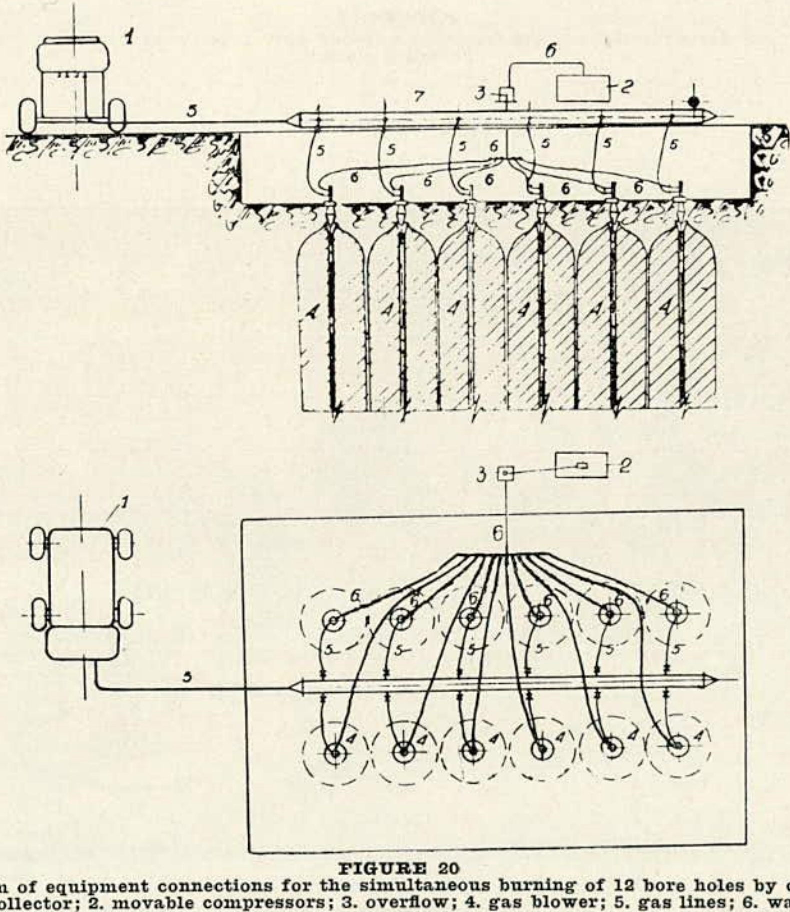

FIGURE 20
Diagram of equipment connections for the simultaneous burning of 12 bore holes by coke gas. 1. Air collector; 2. movable compressors; 3. overflow; 4. gas blower; 5. gas lines; 6. water line; 7. gas collector.

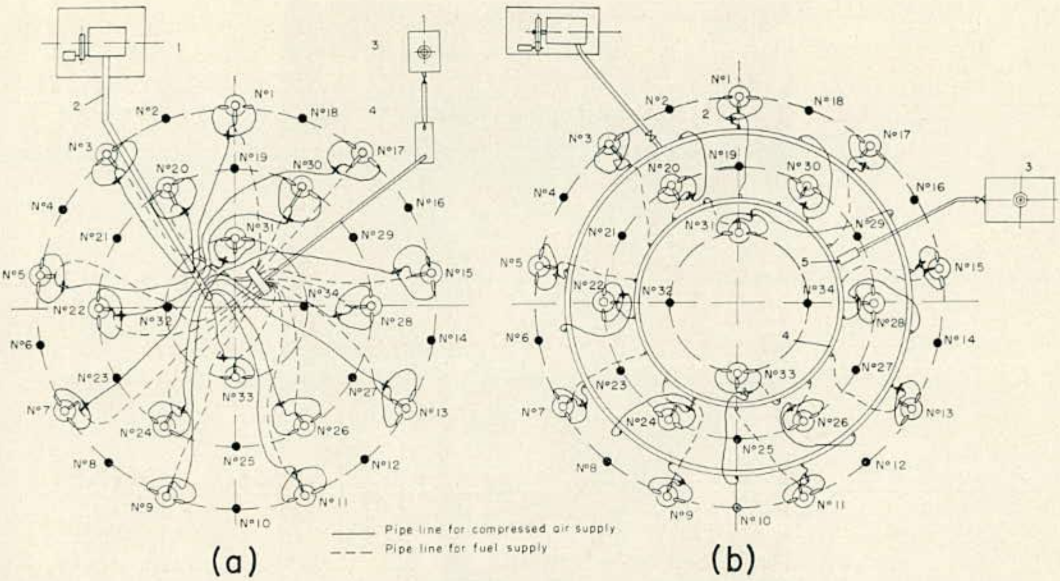

FIGURE 21

Diagram of equipment connections for simultaneously burning of 17 bore holes by liquid fuel. (a) First alternative (using benzo-resisting distribution hoses); 1. compressor; 2. receiver for compressed air; 3. overflow tank for solar oil; 4. fuel container. (b) Second alternative (partial replacement of benzo-resisting hoses by metal pipes); 1. compressor; 2. receiver for compressed air; 3. overflow tank for solar oil; 4. fuel receiver; 5. pump.

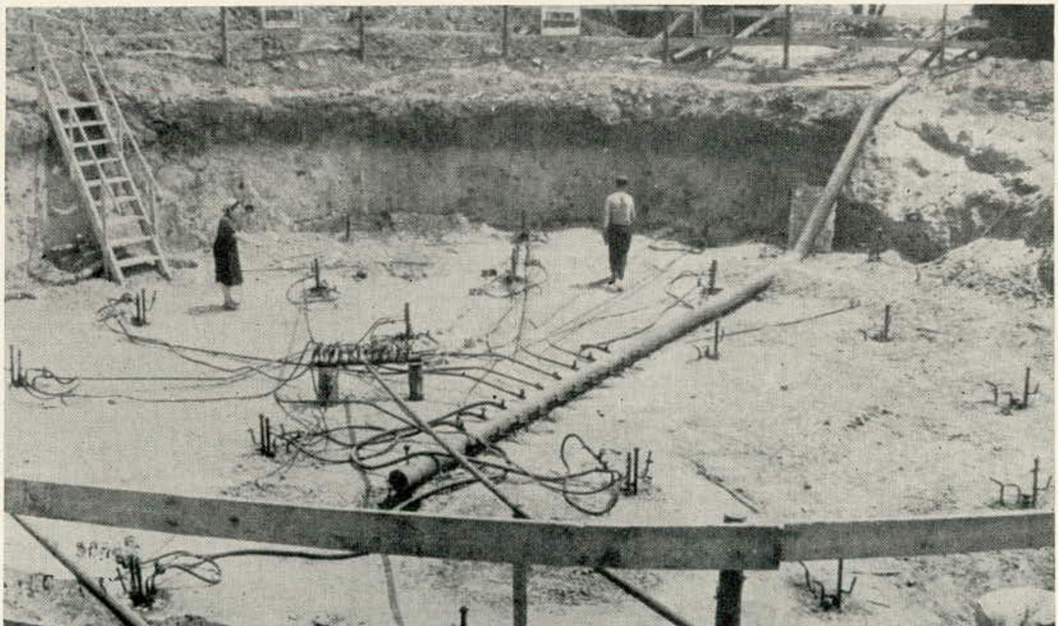

FIGURE 22

View of pit for 100-meter high chimney with foundation underpinned by thermal treatment. Bagley Coke Chemical plant.

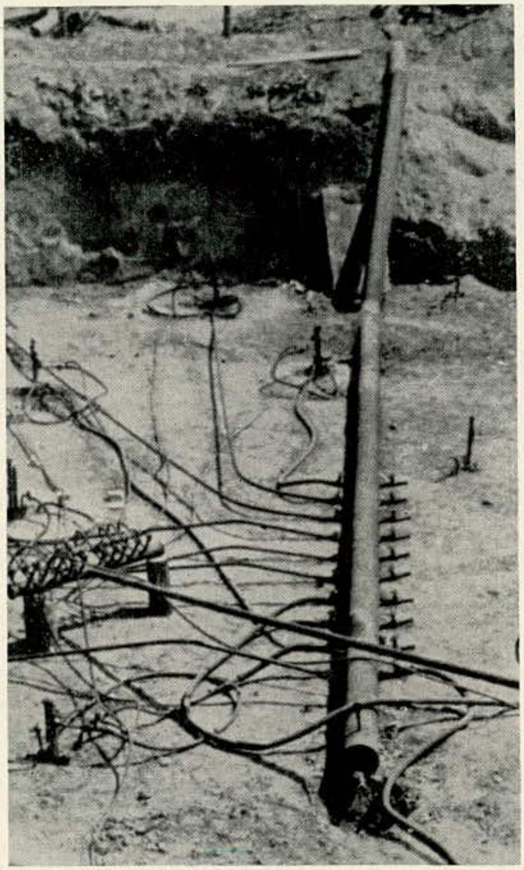

FIGURE 23
Same site with bore holes connected to air collector.

FIGURE 24
Foundations of this house in Zaporozhye were thermally underpinned (1955).

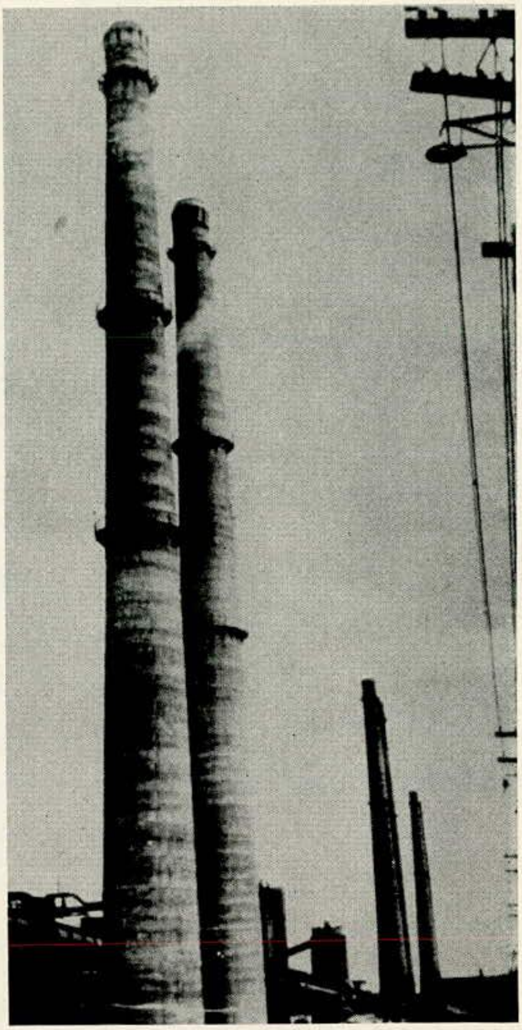

FIGURE 26
Foundations for these 100-meter chimneys in Dnieprodzerzhinsk were thermally underpinned.

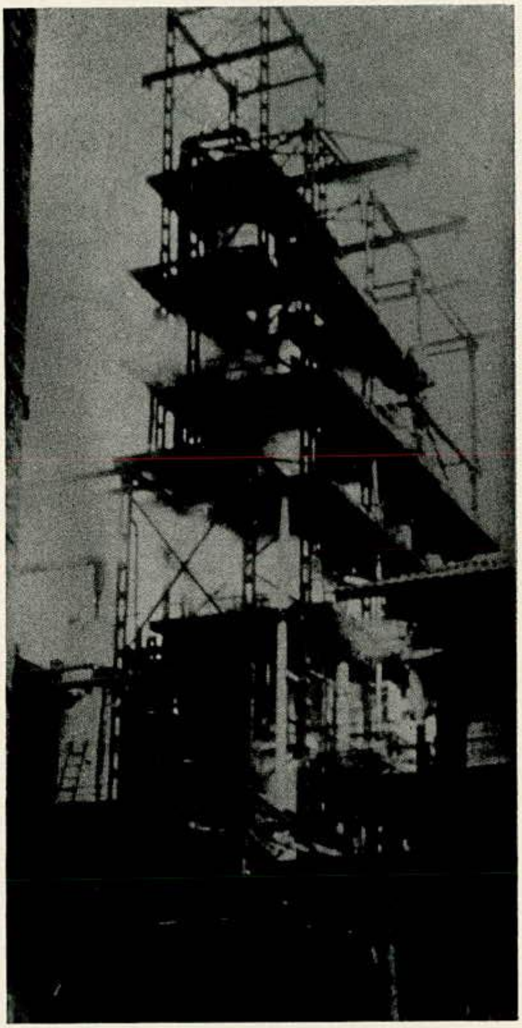

FIGURE 25
Ground beneath this structure was thermally treated (1957).

FIGURE 27
Foundations of this building in Zaporozhye showed no additional settlement when flooded with storm water.

erected on weak soils requiring treatment *(Figs. 24, 25, 26, 27)*.

The new method of loess soil consolidation has aroused great interest among the builders not only in the Soviet Union but in other countries as well.

At the Fourth International Congress on Soil Mechanics and Foundation Engineering in London in August 1957, the author made a report on the thermal treatment of settling loess soils, which was then published in the proceedings of this Congress and in a number of technical magazines in other countries.

Foundation Analysis for Machines with Dynamic Loads

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Since 1925, as a result of extensive development of large-scale industrial construction, systematic studies of the dynamics of machine foundations have been carried on in the USSR. These studies were conducted both by experimental and theoretical methods.

A characteristic feature of the experimental work was that it was performed, not in laboratories but directly on construction sites, and data thus obtained were checked by investigating numerous foundations at industrial plants in operation.

These data, combined with the results of fundamental research, enabled us to evolve sufficiently reliable and practical methods of foundation analysis for machines with dynamic loads.

The scope of the present paper precludes the possibility of dwelling on details of the experimental and theoretical research and I will confine myself to a description of the methods of machine foundation analysis which have been established and checked by the practice of many years.

The methods of machine foundation analysis are controlled by specifications evolved by the Foundation and Underground Construction Research Institute with the participation of several research and designing agencies. As far as we know, such specifications applying to the foundations of a large range of machinery exist only in the USSR.

According to the latest edition of the specifications approved by the USSR State Building Board and made mandatory from July 1958, machine foundation analysis includes the check of the following items:

(a) Strength of the foundation soil.
(b) Strength of the foundation structure.
(c) Amplitudes of foundation vibration.

The results of the analysis serve as a check and, whenever necessary, for specifying the principal dimensions, which, however, are chosen in most cases from design considerations (a suitable location of the machine and its secure installation on the foundation).

(a) Foundation Strength Check

This check of foundations for machines with dynamic loads is carried out in the same way as for foundations carrying static loads: the average soil pressure with respect to the base of the foundation exerted by the weight of the machine and of the foundation structure, is compared with the design resistance of the soil under the foundation.

The only difference from static design is the selection of the design resistance of the soil. A dynamic load is taken into account through reducing the design resistance, in some cases according to the formula:

Where R_d is the design resistance of the machine foundation base; R_{st} is the design resistance of the base (bed) according to the standards for static loads; α is the empirical reduction factor, the value of which depends on the type of machinery according to the table below.

$$R_d = dR_{st}$$

Type of Machinery and Equipment	α
Crank shaft machines, rolling and crushing equipment, metal-cutting machine tools	1.0
Turbine-driven sets, electric machines, grinding installations	0.8
Forge hammers	0.4

(b) Foundation Structure Strength Check

1. The check of the strength of a foundation is carried out, whenever necessary, according to the conventional rules of structural mechanics. The relative permanent and temporary loads characteristic of the machine under discussion are introduced into the computations.

The strength check for turbine-driven sets and electric machines, for example, is conducted for the following design loads:

(a) Permanent loads depending on the machine weight (including the weight of the revolving parts), the auxiliary equipment weight and the dead load of the construction elements.

(b) Temporary loads corresponding to the dynamic effect of the machine, the vacuum suction (draft) in the condensor, the moment of short circuit as well as the assembly loads.

2. The temporary design loads corresponding to the dynamic effect of turbine-driven sets and electric machines acting at right angles to the machine shaft are determined, regardless of the capacity, according to the formulas:

$$N_1 = \gamma_1 G_r \text{ and } N_2 = \gamma_2 G_r$$

where N_1 and N_2 are vertical and horizontal design loads respectively

G_r is the weight of the revolving parts for the given foundation element.

γ_1 and γ_2 are the factors accounting for the alternating effect of the loads and selected according to the table below:

Rpm	For Vertical Loads γ_1	For Horizontal Loads γ_2
1500 and more	15.6	2.6
Less than 1500 and down to 500	10.4	2.6
500 and less	5.2	2.6

3. The temporary design loads corresponding to the vacuum suction in the condensor P_t and the short circuit moment, are usually supplied by the manufacturer of the machine. If these data are lacking, the following empirical formulas are sometimes used to determine the above-mentioned loads.

$$P_t = 11F \text{ (in tons)}$$

and

$$M_3 = 4W \text{ (in mm)},$$

where:

F is the area of cross-section of the neck coupling the condensor with the turbine in sq m.

W is the generator capacity in thousands of kw.

4. The forces acting on the foundation elements of turbine-driven sets and electric machines are determined for the following combinations of loads:

(1) Primary combinations of loads consisting of permanent loads, the load from the vacuum of the condensor P_t and of one of the temporary loads N_1 (directed downward) or N_2.

(2) Special combination of loads consisting of the loads of one of the primary combinations and the short-circuit moment.

(c) Foundation Vibration-Amplitude Check

1. To find a sufficiently rigorous solution of the problem of vibrations of massive foundations for machines with dynamic loads we would have to consider a rather complex system consisting of several elastic related bodies. These bodies—the parts of the machine, the foundation block, and the mass of the soil in the foundation base differ considerably from each other in shape, dimensions and elastic properties.

As was suggested by N. P. Pavlyuk (1933), the following substantial simplifications were adopted for the solution of this problem:

(a) The machine and the foundation are regarded as absolutely rigid bodies.

This simplification proves to be completely justified if one takes into consideration that the dimensions of the machine and foundation are small if compared with the dimensions of the effective zone of the base, whereas the values of the elasticity modulus of metal and concrete are hundreds and thousands of times greater than the values of the elastic moduli of sand and clay.

(b) The base of the foundation is assumed to be elastic and void of mass.

This second assumption is more dubious from the theoretical point of view. As was shown by numerous experiments by Barkan, Pavlyuk and other researchers, the estimates based on this assumption yield results which are in good agreement with reality, provided the elastic characteristics of the base are correctly selected. The research carried out by O. Ya. Shechter in 1948 also showed that the neglect of the inertial properties of the soil does not result in any essential distortions in the computations.

Thus, the dynamic analysis of massive foundations may be based on the consideration of the problem of the oscillations of a solid body resting on an elastic base represented by a model of springs.

2. As was pointed out, the correct selection of the elastic characteristics of the foundation base (factors of its rigidity) is one of the most important conditions for obtaining satisfactory results.

As is known, factors of rigidity of the foundation base may be determined by the formulas:

$$K_z = C_z F; \ K_x = C_x F \text{ and } K_y = C_y T$$

where K_z, K_x and K_y are the factors of rigidity of the base for elastic uniform compression, elastic uniform shear and elastic non-uniform compression, respectively.

F and T are the area of the foundation and the moment of inertia of its base with respect to the main axis normal to the plane of movement.

C_z, C_x and C_y are the dimensional factors of proportion for the corresponding types of elastic deformations of the base.

To determine the factors C the method of local deformation was used for a long time. The method was based on the Fuss (Winkler) hypothesis according to which the normal component of specific pressure at any point of the bases is directly proportional to the local elastic deformation at this point.

As was shown by experimental research, the factors are not permanent values and depend, not only on the elastic properties of subsurface, but on other data, and primarily on the shape and size of the foundation base.

The data obtained made it imperative to give up the Fuss hypothesis. Therefore, Prof. Barkan of the Foundation and Underground Construction Institute, suggested in 1934 that the model of a weightless elastic isotropic semi-space should be used for these purposes. On this basis sufficiently simple expressions were obtained for the coefficients C, which are similar to the Schleicher equation for a rigid die.

The new theory corresponds better to reality. However, here too we find some deviations. According to the new theory the coefficients C decrease in inverse proportion to the square root of the area of the foundation. However, according to experimental results, the decrease of C proceeds at a somewhat slower rate. The deviation between theory and experiment increases with the size of the foundation area. Therefore, in design practice it is recommended to utilize theoretical relationships for foundations with a base area F equal to or smaller than 10 square meters; for larger areas the coefficient C is assumed to be constant. As a rule the coefficient C has to be determined by trial load tests of the soil.

3. According to the technical regulations of the Soviet Union the foundation design for oscillations is compulsory only for forge hammers, jaw crushers and crank-shaft machines in the presence of exciting forces of the first harmonics, as well as for electrical machines with less than 1,000 rpm (forced oscillations).

Computations are made for the purpose of comparing the magnitudes of oscillation amplitudes which are expected with the values which are permitted for a given type of machine.

The study and the analysis of the operation of machines in actual practice have made it possible to establish rules for the permissible magnitude of the amplitudes of oscillation. These values are:

(1) For crank-shaft machines—
with less than 200 rpm....0.25 mm
with 200 to 400 rpm......0.20 mm
with more than 400 rpm...0.15 mm

(2) For gyration and jaw crushers ....0.30 mm

(3) For electric machines—
with less than 500 rpm....0.20 mm
with 500 to 750 rpm......0.15 mm
with more than 750 rpm...0.10 mm

(4) For forge hammers—
with saturated sands in the base ....0.80 mm
with other soils...........1.20 mm

4. The practical methods of the determination of the amplitudes of foundation oscillations are based on a well-developed theory of the oscillation of a solid body resting on the elastic base mentioned above. This theory has incorporated certain simplifications and specifications following numerous experiments carried out in the USSR by D. D. Barkan, A. D. Kondin, N. P. Pavlyuk, O. A. Savinov, Y. N. Smoshkov and others.

In this short paper there is no need to set down formulas for foundation analyses of various machines since these formulas have been extensively cited in print.

On the Design of Foundations by the Method of Limit State of the Soil Base

NIKOLAI A. TSYTOVICH

The method of foundation design according to the limit state of the soil bases is used at present in the USSR and was developed on the strength of the latest data on soil mechanics as well as the practice growing out of many years of observations of settlements and deformations of structures. These data showed that the bearing capacity and the deformations of the base depend not only on the soil properties but on the design features of the structures as well.

The method of foundation design according to limit conditions permits:

(a) Full utilization of the strength and deformation characteristics of the soils of the base and a guarantee of the safety of the structures erected, and

(b) Detailed evaluation of separate factors which affect the performance of the base and the foundations by introducing coefficients: of overloading (greater than unity); of uniformity of the foundation (smaller than unity); and of the conditions of work of the foundations, or to be more precise, of the reliability of design assumptions.

The use of this method of foundation design permits considerable economies. Thus, according to data of the Institute of Foundations (Moscow), the use of this method in areas of permanently frozen soils gives an economy up to 15 per cent of the value of the foundations which, over a period of five years, equals approximately one billion roubles.

The following limit conditions of soil bases are considered: limit stability, i.e., the loss of bearing capacity, and limit deformations, i.e., such deformations which when reached cause a rupture of continuity in structures (i.e., the appearance of cracks) so that the use of the structures is impaired.

The design for the first limit condition, the loss of bearing capacity, is based on the general theory of limit equilibrium for which a number of rigorous solutions have been obtained during the last decades both in the USSR, work of V. Sokolovsky, V. Berezantzev and others, as well as in the West, by Skempton, Meyerhoff and others. This method permits the determination of the maximum possible loads on the soil of the base, by using known physical properties of the soil and given boundary conditions.

Two cases have to be considered during such studies:

1. The beginning of appearance under the edge of foundations of zones of limit stress conditions (i.e., under pressures which reach the end of the phase of consolidation—p_o kg/cm^2) *(Fig. 1)*. A full development of zones of limit equilibrium *(Fig. 2)* whereby under the foundation surfaces of slippage of one part of the soil along the other are formed under full (maximum) utilization of the bearing capacity of the foundation. Under such conditions, the slightest increase of the acting load creates a state of unstable equilibrium in a certain zone of the soil under the structure so that a phase of squeezing out of the soil is reached. Thus a design according to the first limit condition permits the determination of the start of plastic flow and of the maximum limit load on the foundation.

Computations according to the second limit condition (for limit deformations) are of basic importance for the design of foundations of structures. The following two conditions serve as starting point for the computations. They vary depending on the uniformity of the soils of the base:

(a) For relatively uniform bases

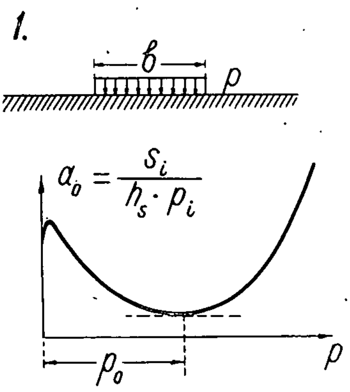
$$S_c < S_u$$

(b) For any types of non-uniform soil bases

$$\Delta S_c < \Delta S_u$$

where S_c is the computed settlement of the foundations determined according to well-known equations of soil mechanics, e.g., according to data given *(Figs. 3, 4)*; and where S_u is the limit settlement of founda-

1.



Fr < 3%	$h_s = 1{,}0 \cdot b$
3 – 10%	$1{,}1\ b$
10 – 30%	$1{,}2\ b$
Fr > 30%	$h_s = 1{,}3\ b$

2.

$$p_u \leqslant \frac{\pi(\gamma h) + \frac{c}{tg\varphi}}{ctg\varphi + \varphi - \frac{\pi}{2}} + \gamma h$$

$$p_u = \pi c_e$$

$$[p_u]_s \leqslant \frac{B}{b} \cdot p_0 ;$$

$$[p_u]_{cl} \leqslant 1{,}2\, p_0 ;$$

FIGURE 1

Determination of the limit of the consolidation phase: 1. according to the results of a load test (determination of the value p_0 kg/cm²); 2. according to theoretical equations (of Pouzirevsky, Froelich and others): determination of the pressure (p_u) below which zones of limit equilibrium are not present in the soil. Where: ψ = angle of internal friction; c = cohesion; B = width of the foundation; b = width of the tested area; a_0 = reduced coefficient of consolidation; c_e = permanent cohesion for cohesive soils; $(p_u)_s$ for sands; $(p_u)_{cl}$ for clays.

tions permissible from the point of view of the strength and of the use of structures—determined according to rules of building codes or on the basis of generalized observations of settlements of structure.

ΔS_c and ΔS_u are the differential settlements of adjoining foundations, or the slopes of their inclined lower surfaces or the relative deflections of continuous strip foundations.

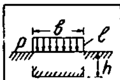
	1.	2.			
	$c = f(t, W, \theta°); \varphi = 0$	h/b	$c=0; \varphi = f(t,W,\theta°); p_u = D \cdot \gamma b + \gamma h$		
			$\varphi = 26°$	30°	36°
$\frac{l}{b} > 10$	$p_u = 5{,}14c + \gamma h$	0	6,8	D = 10,8	26,2
		1	21,3	34,8	79,5
		2	36,3	58,9	138,0
$\frac{l}{b} = 1$	$p_u = 5{,}7c + \gamma h$	0	—	17,3	~56
		0,5	—	33,7	~106
		2	—	122,0	~335
$\frac{l}{b} > 10$	$p_u = c \cdot ctg\varphi(\zeta - 1) + \gamma h \cdot \zeta;\ \zeta = e^{\pi tg\varphi} tg^2(45° + \frac{\varphi}{2})$; $\varphi \neq 0;\ c \neq 0$				

FIGURE 2

Equations for determination of limit loads on cohesive and granular soils: 1. for cohesive clay soils (after Prandtl and Ishlinski); 2. for dense non-cohesive soils (γ = unit weight of soil above base of foundation; h = depth of foundation base below soil surface.

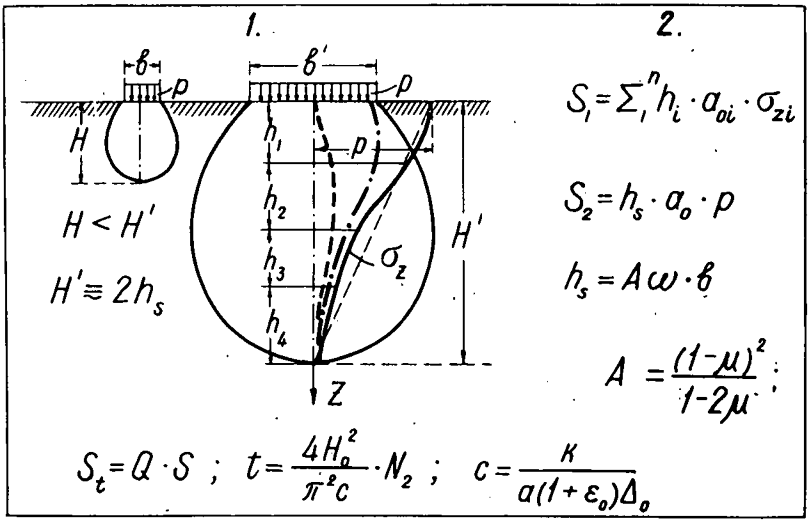


FIGURE 3

Equations for computation of foundation settlements on uniform soils: 1. dependence of zone of consolidation on dimensions of loaded area, the concept of the equivalent soil layer, h_s; 2. equations for computation of foundation settlement on uniform soils: S_1 according to method of elementary summation without consideration of lateral yielding of soil; S_2 according to method of equivalent layer (developed by the author) which takes into consideration lateral expansion of the soil (μ), the dimensions and the shape of the foundation as well as its rigidity (the coefficients w, b); A = coefficient which depends on the lateral yield of the soil (values of Aw given in Fig. 4).

$a = \frac{l}{b}$	Gravel		Sands				Plastic Loams		Plastic Clays	
	Hard Clay and Loams				Sandy Loams					
	$\mu = 0.10$		$\mu = 0.20$		$\mu = 0.25$		$\mu = 0.30$		$\mu = 0.35$	
	Aw_c	Aw_m	Aw_c	Aw_m	Aw_c	Aw_m	Aw_c	Aw_m	Aw_c	Aw_m
1	0.568	0.96	0.598	1.01	0.631	1.07	0.687	1.17	0.790	1.34
1.5	0.687	1.16	0.724	1.23	0.764	1.30	0.832	1.40	0.956	1.62
2	0.775	1.31	0.817	1.39	0.862	1.47	0.938	1.60	1.079	1.83
3	0.903	1.55	0.951	1.63	1.003	1.73	1.092	1.89	1.256	2.15
4	0.994	1.72	1.047	1.81	1.105	1.92	1.203	2.09	1.383	2.39
5	1.065	1.85	1.122	1.95	1.184	2.07	1.289	2.25	1.482	2.57
6	1.124	1.98	1.184	2.09	1.249	2.21	1.360	2.41	1.568	2.76
7	1.173	2.06	1.236	2.18	1.304	2.31	1.420	2.51	1.632	2.87
8	1.216	2.14	1.281	2.26	1.316	2.40	1.472	2.61	1.692	2.98
9	1.254	2.21	1.321	2.34	1.393	2.47	1.517	2.69	1.744	3.08
10 and more	1.288	2.27	1.357	2.40	1.431	2.54	1.558	2.77	1.792	3.17

FIGURE 4

Values of coefficient of equivalent layer of soil Aw for computation of foundation settlement on compressible soils: Aw_c—for the corner points of rectangular flexible foundations; Aw_m—for average settlement of rigid foundations.

Ultimate values of settlements

K_r	$\frac{dh}{dt}$ m/year	S cm	$\frac{dS}{dt}$ cm/year	Slope $i \cdot 10^3$	Deflection $f \cdot 10^3$
1	2	3	4	5	6
30÷300	0,3-0,9	10-20	3-8	1,5-4,5	1-3
<30	0,9-1,8	20-40	8-15	4,5-8	3-6
≥300	to 2	to 50	to 20	to 10	—

$$K_r \cong 1{,}7(1-\mu_0^2)\left(\frac{A_0}{p}+a_0\right)\cdot\Omega\cdot E_B\frac{H^3}{L^3}$$

FIGURE 5

Values of ultimate settlements (after Oushkalov) when erecting structures under conditions of permanently frozen ground. 1. Coefficient of rigidity; 2. rate of thawing of soils under structure, meters per year; 3. values of settlements, in centimeters; 4. limit rate of settlement, centimeters per year; 5. limit inclination (slope) of foundations; 6. limit deflection of foundations. (According to USSR building codes permissible settlement values are approximately one-half the above.)

The last two conditions require consideration of the interaction of the soils of the base and of the structures erected on them as well as the determination of the coefficient of accuracy of settlement forecast. Given *(Fig. 6)* are the results of observations and of computed settlements of one of the buildings and given *(Fig. 7)* is the statistical evaluation of a series of similar observations.

In order to satisfy the above conditions, the designer may change the left side of the inequalities given above (i.e., by decreasing or increasing the load on the base); or he may change their right part (e.g., by using more rigid structures or statically determinate systems); or he may simultaneously change both parts of the inequalities in order to bring their values closer together.

The author recommends the following procedure of foundation design according to limit states of soil bases:

1. A preliminary selection of the dimensions of foundations using data of soil bearing capacities recommended by building codes for designs (as determined by the equations given in Figure 1).

2. Determination of the computed deformations (S_c) and their comparison with the limit values (S_u).

Re-evaluations of the loads selected (usually their increase under consideration of the two conditions: $S_c < S_u$ and $\Delta S_c < \Delta S_u$); and the design of the foundations.

Check computations of the pressures along the base of the foundations and of the deformations of the base.

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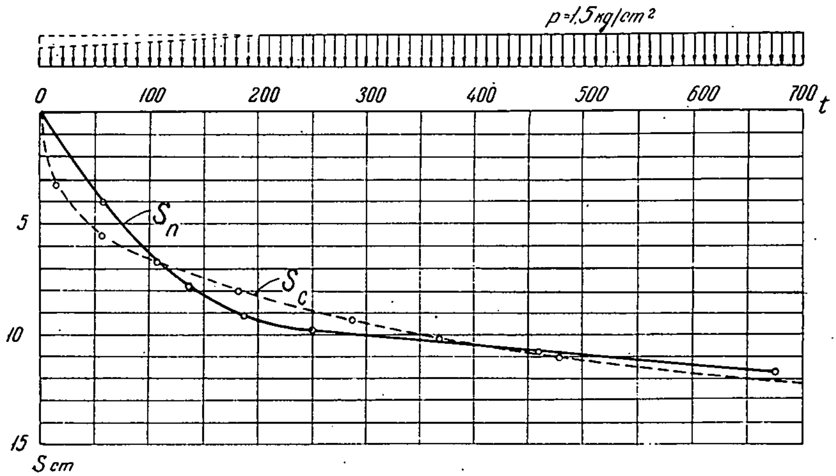

FIGURE 6

Comparison of settlements (S_c) computed according to the method of the equivalent layer with measured settlements (S_n) for a school building erected on a complex (7 layers) soil base.

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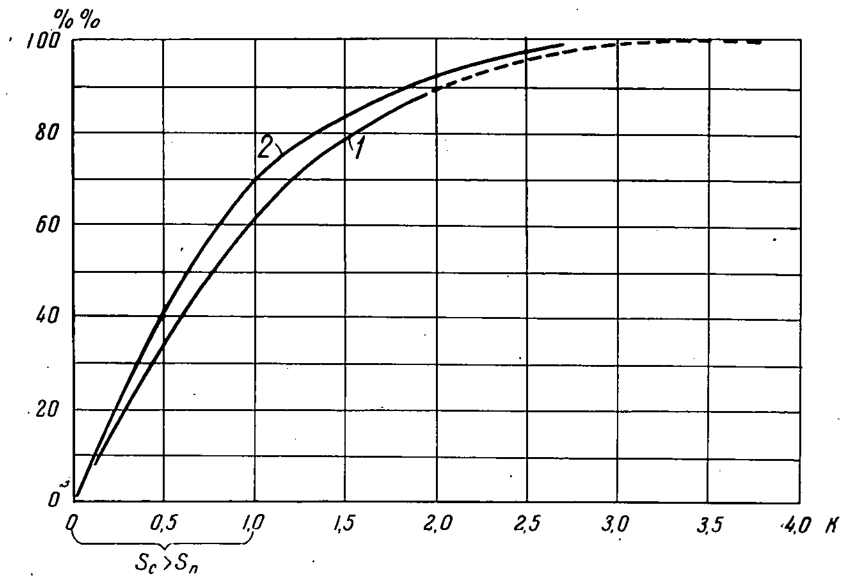

FIGURE 7

Curves of statistical distribution of probability coefficients (K) for various methods of foundation design: 1. when computed by the method of elementary summations; 2. when computed by the method of the equivalent layer of the soil (when 80 per cent of cases of computed settlements are no smaller than the real ones, a probability coefficient varying between 1.3 and 1.5 is obtained).

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The Value of the American-Russian Exchange Visits

I. M. Litvinov, *Chairman,*
The Soviet Delegation
(National Academy of Sciences, Washington, D. C., June 18, 1959)

Permit me in the name of the Soviet Delegation of Scientists and Construction Specialists, to express our cordial gratitude for your amiable invitation to visit your country and to familiarize ourselves with your achievements in the field of theory and practice of soil mechanics and foundation engineering.

We express our gratitude to the National Academy of Sciences, to the American Society of Civil Engineers and to all those gentlemen who with sincere good feeling helped us to utilize our stay in the USA as rationally and usefully as possible with the aim of organizing scientific and business contacts between specialists of our countries.

We particularly thank the deeply esteemed chairman, Dr. Connolly, Mr. Burggraf and Profs. Kersten, Tschebotarioff, Winterkorn, Schmid, Jumikis, Lambe, Peck, Leonards and Seed—who did extensive practical work for the organization of the exchange of our delegations and for the creation of particularly favorable conditions in the USA for our delegation.

Our brief visit to the United States of America is coming to an end. However, although we stayed here a short while, we nevertheless saw and learned must which is interesting and useful. We received much satisfaction from our study of the interesting construction and especially road building sites and test roads in a number of American cities. We were particularly interested by the remarkable organization of test road research. We received much that was interesting and valuable as a result of our participation in seminars concerned with various problems in the field of soil mechanics and foundation engineering.

At these seminars, as well as in laboratories, on construction sites and in other places, we had the agreeable opportunity, not only to present reports concerning a number of our Soviet scientific studies, but also to make personal contacts with many outstanding scientists and specialists of America.

The Soviet scientists and specialists follow with great attention the interesting, and in a number of cases, outstanding achievements of American scientists. In spite of complications in the matter of direct exchanges between the specialists of our two great countries, we in the USSR study with great respect and interest the work of the specialists and scientists who are known the world over in that field: Terzaghi, Casagrande, Tschebotarioff, Peck, Burggarf, Lambe, Osterberg, Kersten, Hanson, Thornburn and many others. In the USSR books of American specialists have been published in large printings.

The considerable achievements of American specialists in the field of road construction are generally acknowledged. In this respect, the American roads designed and built in complicated geological conditions are presentations in nature equivalent to beautiful books and scientific studies.

Much that is useful is being done by Soviet scientists, too. We also have a number of achievements which help the technical advance of construction work, and because of geographical and climatic conditions of our country, we have to develop solutions to numerous engineering-technical problems which do not occur in other countries.

The hydro-technical construction widely developed in the USSR demanded the solution of many complicated problems. These included the solution of the problem of construction on soft soils and on sands of large structures required to resist considerable heads of water pressures.

In June 1956, in Moscow, the Academy of Construction and Architecture of the USSR was created, and in November 1956, in Kiev, the Ukranian Academy of Construction and Architecture was established. In the systems of the various scientific-research organizations of these two academies more than 12,000 scientific and engineering-technical workers are occupied with the complex multi-phased study and development of the most important problems in the field of construction and architecture, which include the fields of soil mechanics and foundation engineering.

It seems to me that many results of Soviet scientific studies would be of interest for American scientists and engineers as well and we are glad to find that scientific and cultural connections between our countries are becoming more and more organized. We are deeply convinced that mutual exchanges of specialists will favor further cultural and scientific progress, and, above all, will help create friendly connections between peoples and will affirm peace.

Often we have been asked what we liked best in America and without hesitation we answered: "The Americans who, just as do we, want a peaceful life and friendly exchanges in business and scientific fields." We are certain that the results of our trip will be useful to our scientific and practical activity; we would like to hope, too, that this visit will in some way prove not without use for you.

Travel from one country to the other took a long time in the past. There are no such difficulties today. Having had lunch in Moscow, the same evening we had supper in New York, so that the technical possibilities for the expansion of association between our countries have now become limitless.

For that reason, we hope, that our mutual visits will promote the development of friendly business and scientific relations between our scientists and specialists. And this will promote not only the development of science and of practice in the field of soil mechanics and foundation engineering, but also the improvement of mutual understanding and friendship between the peoples of our great countries and will contribute to a quiet, peaceful life on our, as yet, insufficiently firm earth globe.

VII — Papers Presented by the American Exchange Delegation in the USSR

The seven members of the United States delegation presented 11 papers at three seminars during the visit to the Soviet Union, May 31-June 21, 1959. Some of these were essentially reviews of work previously presented or published but five of the papers had not been presented in America. They are:

"Floating Caisson Foundations. Steel Piles with Attachments," by John Lowe, III.

"Current Practice in Soil Sampling in the United States," by John Lowe, III.

"Certain Features of Lateral Pressures of Soils," by Gregory B. Tschebotarioff.

"Critical Elements of Design and Construction of Heavy-Duty Flexible Pavements," by W. J. Turnbull.

"A Summary of Rotary Cone Penetrometer Investigations," by W. J. Turnbull, et al.

Brief resumés are included of the subjects presented by Prof. Kersten, "Studies of Frost Problems in a Northern State"; Prof. Lambe, "Soil Stabilization" and "Soil Structure"; Prof. Leonards, "Analysis of Design of Concrete Slabs on Ground"; and Prof. Seed, "Recent Researches in the USA on Soil Strength and Deformation Characteristics Under Dynamic Loading Conditions"; and "Foundations for Large Bridges Across the San Francisco Bay."

It was understood that all of these papers were to be published in the USSR in Russian language translations.

Floating Caisson Foundations

JOHN LOWE, III, *Associate Partner,*
Tippetts, Abbett, McCarthy, Stratton

In recent years two important structures in the New York City area, Pier 57 and the Nyack-Tarrytown Bridge, have been founded upon a new floating caisson type foundation. About 85 per cent of the load of the pier and 65 per cent of the load of the bridge are carried by buoyancy.

The development of this type of foundation and the design of the structures were carried out by Capt. E. H. Praeger, U. S. Navy Retired, who has kindly made available photographs and other material to the author for the preparation of this paper. The designs of the foundations of the two structures are described below starting with the bridge.

NYACK-TARRYTOWN BRIDGE

The Nyack-Tarrytown bridge across the Hudson River is an important link in the New York State Thruway system which extends 400 miles from New York City to Buffalo, and is presently being extended an additional 100 miles westward from Buffalo along the shores of Lake Erie. The minimum roadway design consists of two north-bound lanes and two south-bound lanes with a wide median strip. In the vicinity of New York City and other areas of heavy traffic, three or more lanes are provided in either direction. Not a single grade intersection or toll booth interrupts the flow of high speed traffic from the bridge for the full length of the Thruway.

The crossing of the Hudson River was chosen at Nyack-Tarrytown on the basis of traffic patterns and topography on both sides of the river as well as a territorial franchise held by The Port of New York Authority, operators of the George Washington Bridge across the Hudson River at New York City, which prohibits the construction of a toll bridge south of the

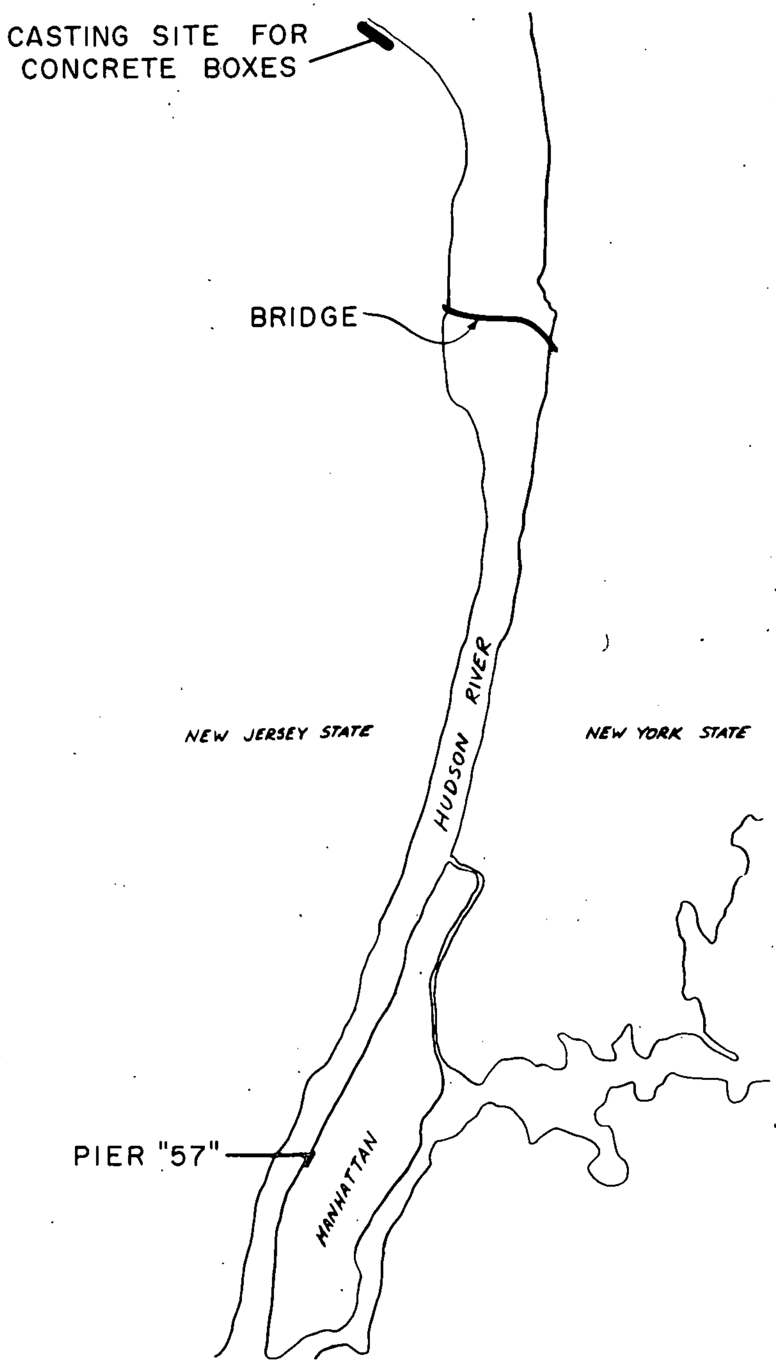


FIGURE 1
Location plan of Tappan Zee Bridge.

Nyack-Tarrytown location. The location of the Nyack-Tarrytown bridge is shown *(Fig. 1)*. The location of Pier 57 is also shown on this figure.

The bridge consists of three types of superstructure construction as may be seen in the aerial view of the bridge *(Fig. 2)*. The total width of the Hudson River at the site is three miles. Across the main river channel is a 1,212-ft cantilever span with two 602-ft anchor spans; on either side of the main spans are a total of 19 deck truss approach spans each of which is approximately 250 ft in length; from Nyack on the west shore, toward the center of the river is a trestle type structure approximately 8,000 feet in length and having 50-ft spans. A closer view of the main spans is given *(Fig. 3)*. The 1,212-ft main span was dictated by clearance requirements. The 602-ft anchor spans are structural adjuncts of the main span; the trestle spans were dictated by foundation conditions.

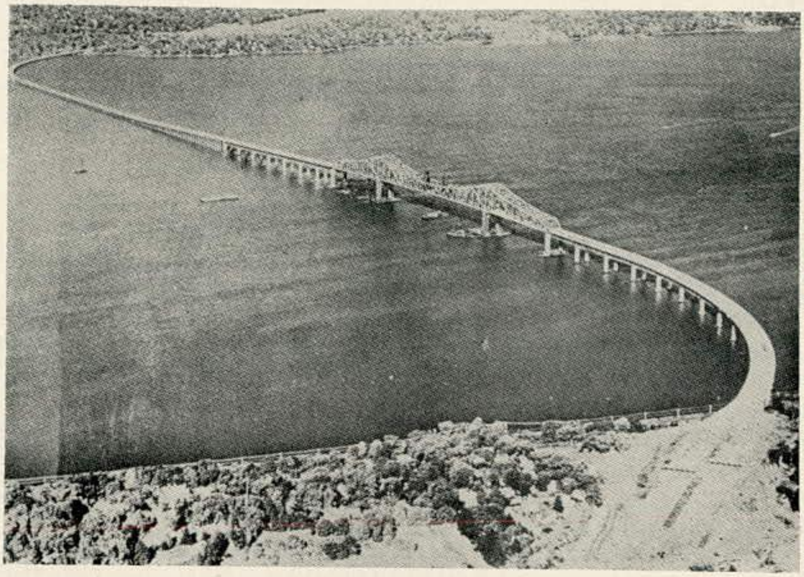


FIGURE 2
Tappan Zee Bridge, aerial view.

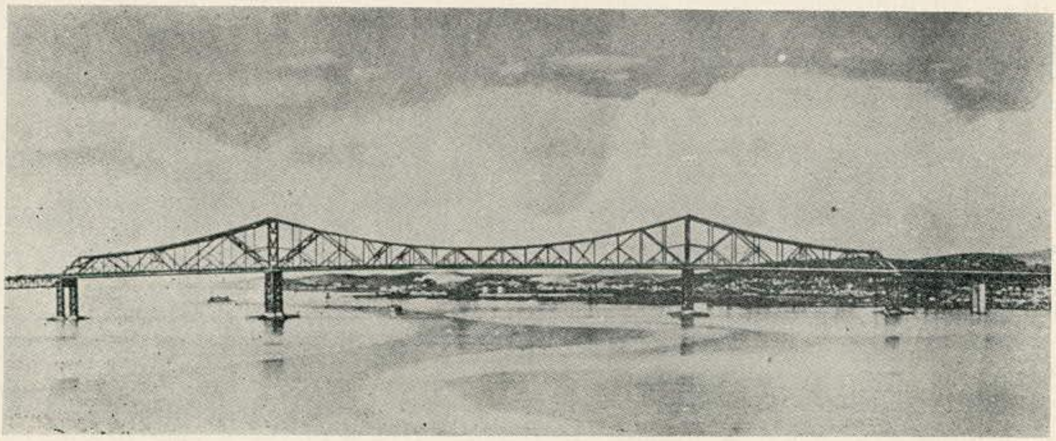


FIGURE 3
Main spans of Tappan Zee Bridge.

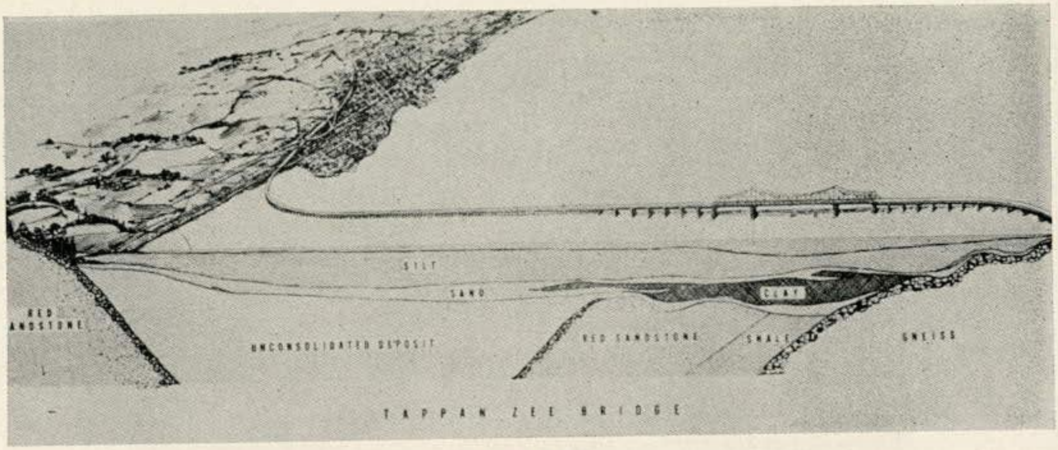


FIGURE 4
Geologic profile.

A geologic profile at the site is shown *(Fig. 4)*. Borings were made by barge mounted drill rigs. Testing of undisturbed samples was carried out in the soils laboratory of Columbia University. Gneiss bedrock was found to outcrop at Tarrytown on the east shore and dip gradually to the west. Overlying this basement rock, sandstone and a shale hardpan were encountered. These strata also dip to the west and at the center of the river occur at depths exceeding 300 ft. Water depths are 10 to 12 ft over the westerly half of the river and soft to medium stiff silt extends for depths of several hundred feet below river bottom. The strength of the upper silt is too low to permit construction of an approach embankment. The solution adopted utilizes concrete bents spaced 50 ft on centers and supported on 80-90 ft long untreated peeled Douglas Fir timber piles. Design loads are 8.5 tons dead plus 4 tons live.

Fifteen of the 19 approach spans are supported on concrete piers each of which consists of a two-legged bent founded on steel H-piles driven to rock.

The four towers supporting the main span and the anchor spans as well as four piers supporting approach spans on the westerly side are founded upon the floating caisson type foundation. Here the river bottom is 40 ft deep and sandstone bedrock occurs at a depth of as much as 260 ft below river bottom.

The floating caisson type foundations consist of pre-test hollow concrete boxes. The boxes were designed to carry 65 per cent of the dead load of the structure by buoyancy. The remaining dead load and all live load is carried by steel piles driven to rock. Fourteen-in. steel H-bearing piles weighing 89 lbs per ft were used in the case of the approach span piers and 30-in. diameter steel pipe piles, with a wall thickness of ½ in. were driven for the main span and anchor span piers. The pipe piles were driven open ended, cleaned out, and filled with concrete. The piles are connected to the caisson boxes by driving them through holes cast in the walls of the boxes and concreting the holes. The foundation is illustrated *(Fig. 5)*.

The eight boxes, one for each of the above mentioned piers, were prefabricated in a construction basin at West Haverstraw, approximately 12 miles north of the bridge site. An aerial view of the basin filled with water is shown *(Fig. 6)*. Equipment used for dewatering the basin is shown *(Fig. 7)*. Later views of the basin in the dry condition and of the eight boxes under construction are shown *(Figs. 8, 9)*. All boxes are 40 ft deep; the two boxes supporting the main span are 100 ft wide by 190 ft long; those supporting the anchor spans are 77 ft wide by 124 ft long; and those supporting the approach spans are 56 ft wide by 110 ft long. All boxes have interior longitudinal and transverse walls which divide them into

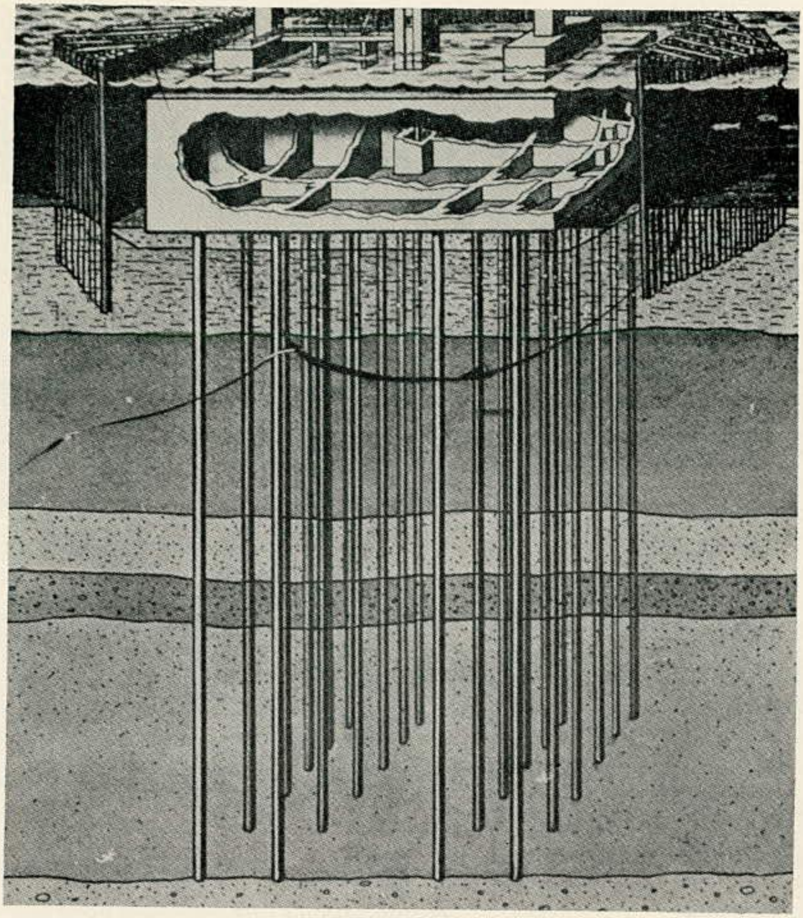


FIGURE 5
Floating caisson foundation.

compartments, and an intermediate floor under the top deck. The largest boxes weigh 23,000 tons each and have a draft of 30 ft. The top sections of the walls and the tops of the boxes were cast at the bridge site.

A view of the reinforcement for the boxes and the corrugated metal pipes forming the holes for the piles is shown *(Fig. 10)*. Reinforcing steel up to 2-in. square bars were spliced by the Thermit Weld process. A view of the splicing is shown *(Fig. 11)*. Another view of the details of the boxes is shown *(Fig. 12)* and a closeup of the completed boxes *(Fig. 13)*.

Concrete was made with carefully selected and proportioned aggregate consisting of both crushed basalt and quartz sand and gravel. Six bags of cement were used per cu yd. The concrete was placed at low slump, vibrated, cured first with water, and then cured with a coating consisting of a mixture of asphalt and aluminum powder. The aluminum was incorporated into the compound to reflect the heat of the sun. Strengths up to 4,000 lbs per sq in. at seven days and up to 7,000 lbs per sq in. at 28 days were obtained. Both vertical and horizontal pours terminated in cold joints and these as well as shrinkage cracks were sealed with a Neoprene process developed for this job. The testing of a joint for water tightness is shown *(Fig. 14)*. A special chamber was clamped to the wall, and pressure built up against the wall.

After completion of the casting of the eight boxes, the basin was flooded, the dike along the Hudson River breached and the boxes towed one by one to the site. Shown

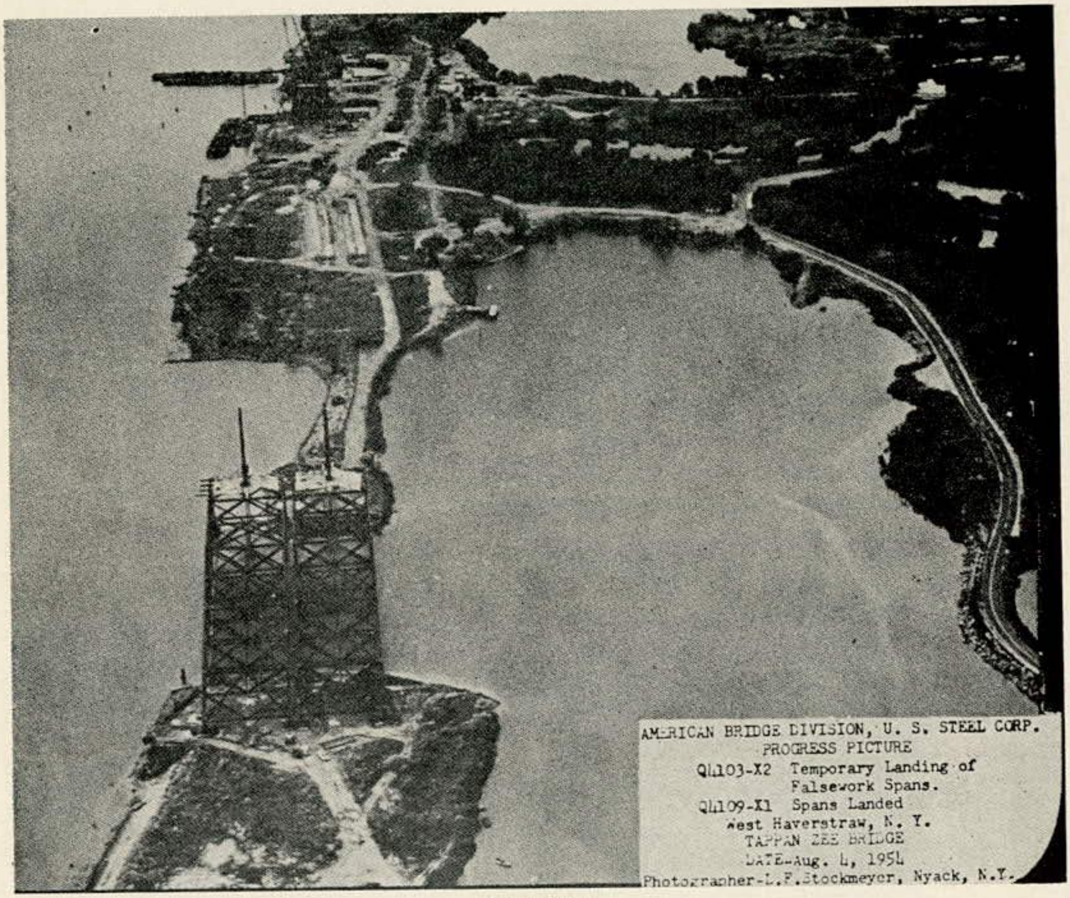


FIGURE 6
Construction basin for prefabricated foundation caissons.

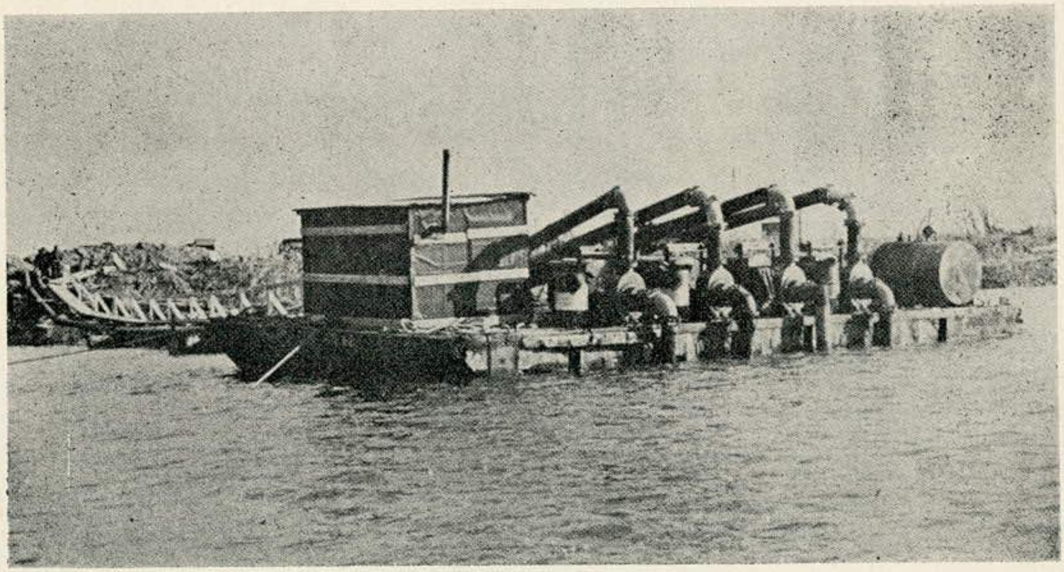


FIGURE 7
Dewatering equipment for construction basin.

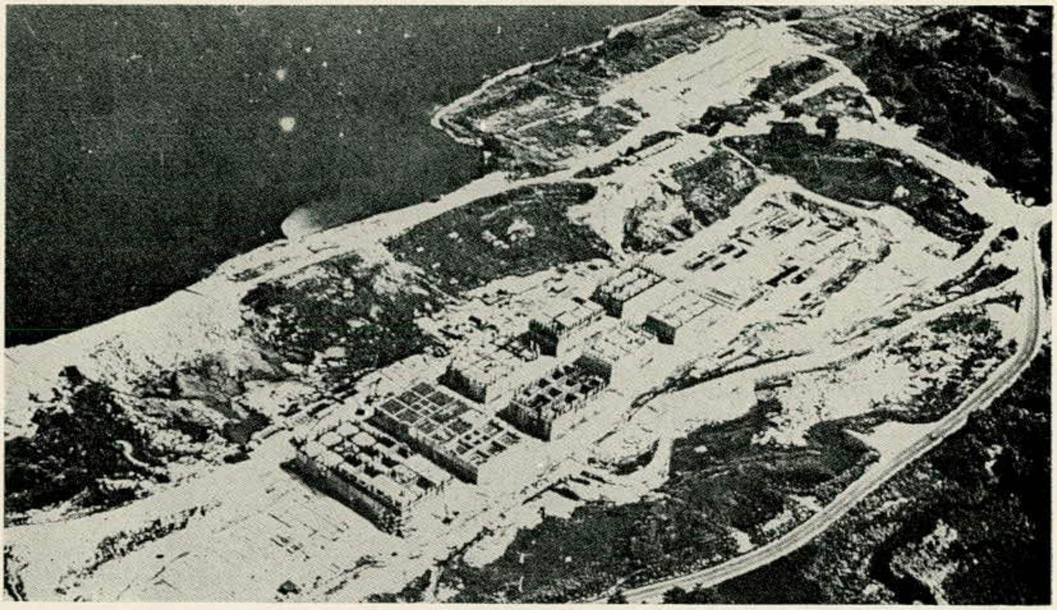


FIGURE 8
Foundation caissons under construction in the dry.

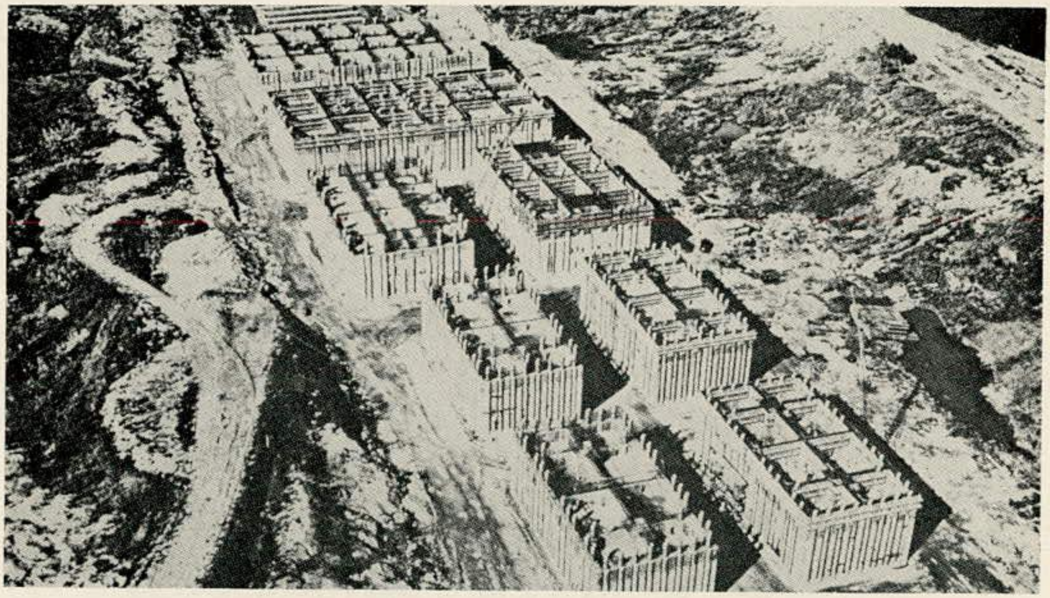


FIGURE 9
Foundation caissons under construction in the dry.

(Fig. 15) is the first box being towed out of the basin. Shown *(Fig. 16)* is the box en route to the bridge site and illustrated *(Fig. 17)* is the box held in position by clusters of timber piles.

When the walls and tops of the boxes were completed at the site the boxes were sunk by partially filling them with water so that their top was below extreme low water level. The piles were then driven and cast into the

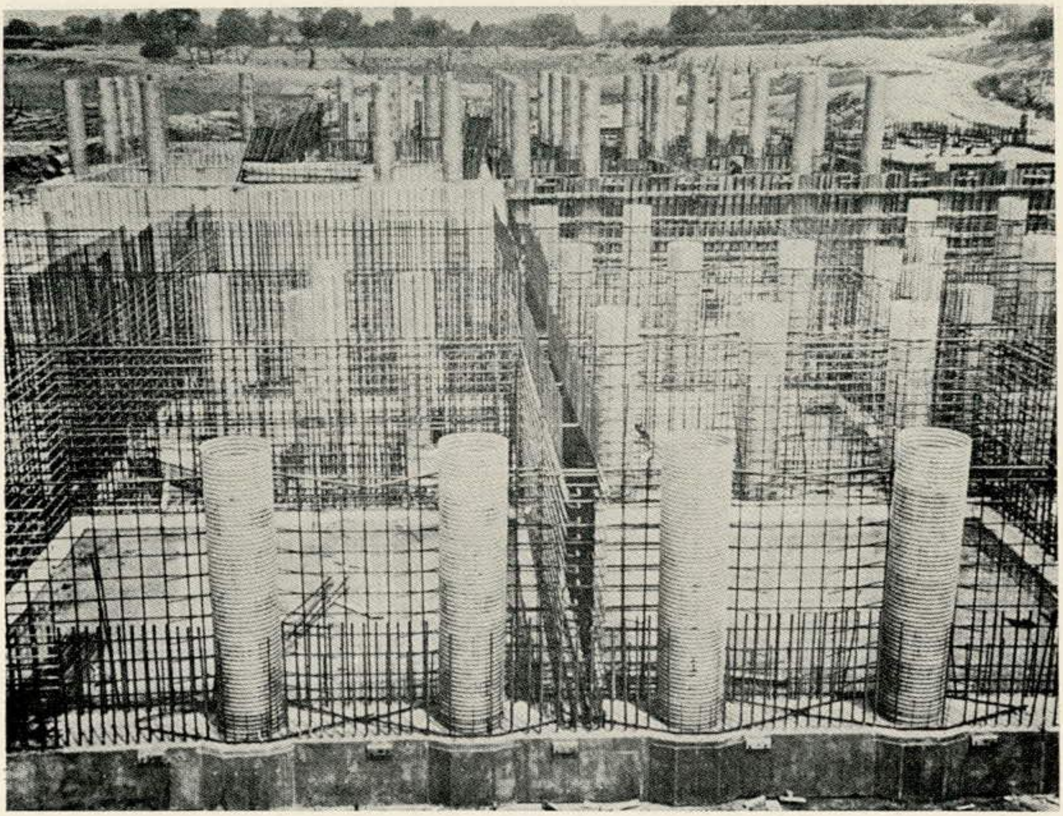


FIGURE 10
Caisson reinforcing bars and corrugated metal forms for holes in piles.

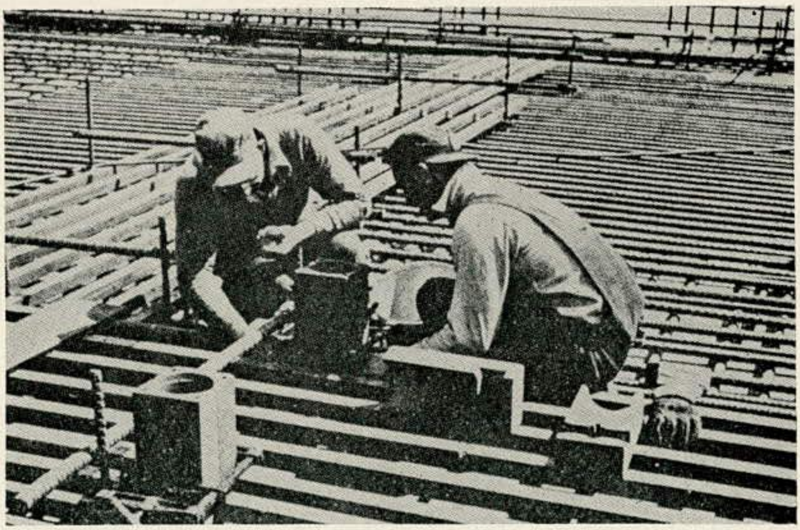


FIGURE 11
Splicing of caisson reinforcing bars by Thermit weld process.

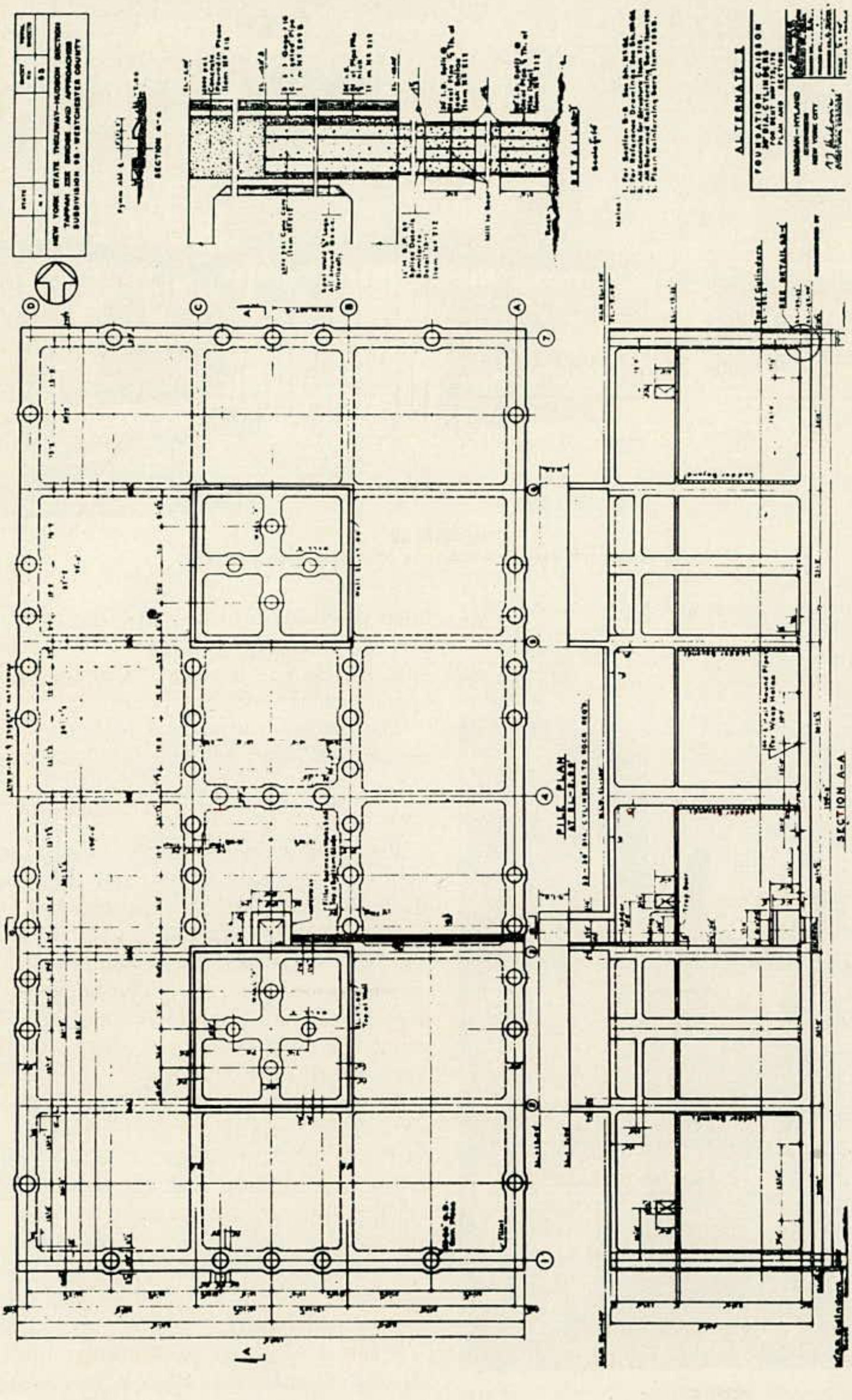


FIGURE 12
Foundation caisson plan and section.

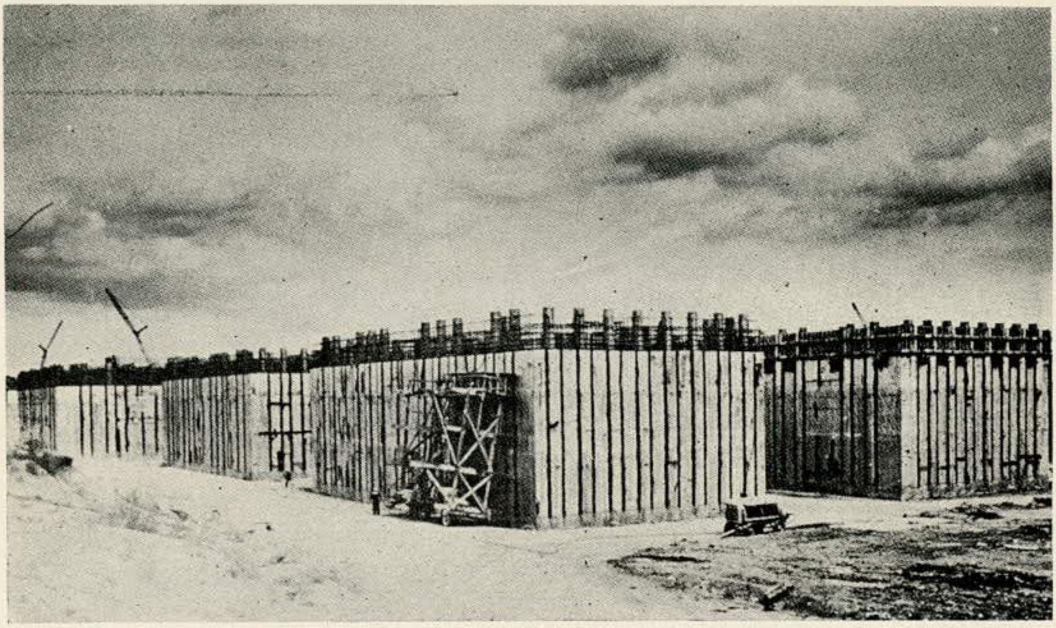


FIGURE 13
Completed prefabricated sections of foundation caissons.

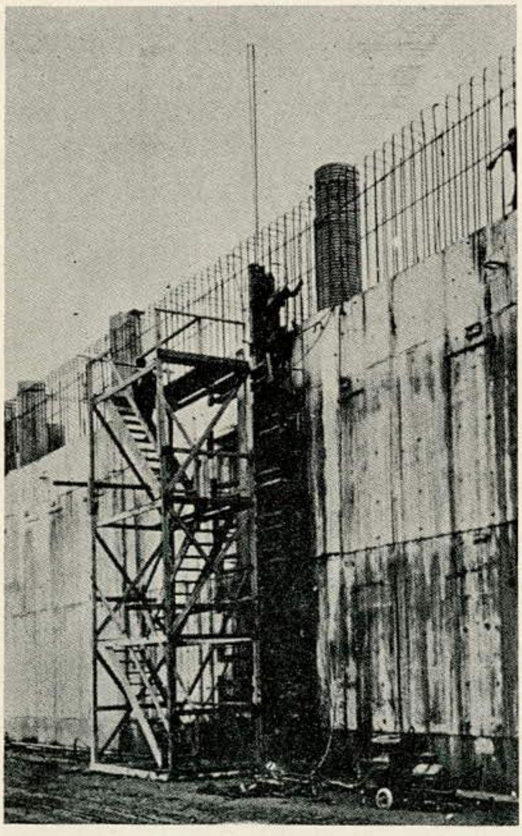


FIGURE 14
Pressure testing caisson joint for watertightness.

holes provided for them. As the superstructure was completed the boxes were pumped out. Access has been provided for periodic inspection of their watertightness.

The bridge was opened to traffic in December 1957 and has operated perfectly.

PIER 57, NEW YORK HARBOR

The pier which formerly existed at the foot of West 15th Street and the Hudson River, New York City, was destroyed by fire in 1947 *(Fig. 18)*. In 1952 a new "T" shaped pier was constructed. A plan of the new pier is shown *(Fig. 19)*. The finger extending into the river is 725 ft long and 150 ft wide; the portion along the shore is 375 ft long and also 150 ft wide.

Wharfage space on the New York City waterfront is at a premium, thus the Department of Marine and Aviation, owners of the pier, desired that the new pier have much more capacity than the old pier. This indicated a pier with several levels and much higher unit loadings than the old structure. Also, it was desired that the new pier be fireproof.

A forest of about 3,000 timber piles was all that remained of the old pier. Subsurface explorations at the site indicate bedrock

FIGURE 15
Towing first foundation caisson from construction basin.

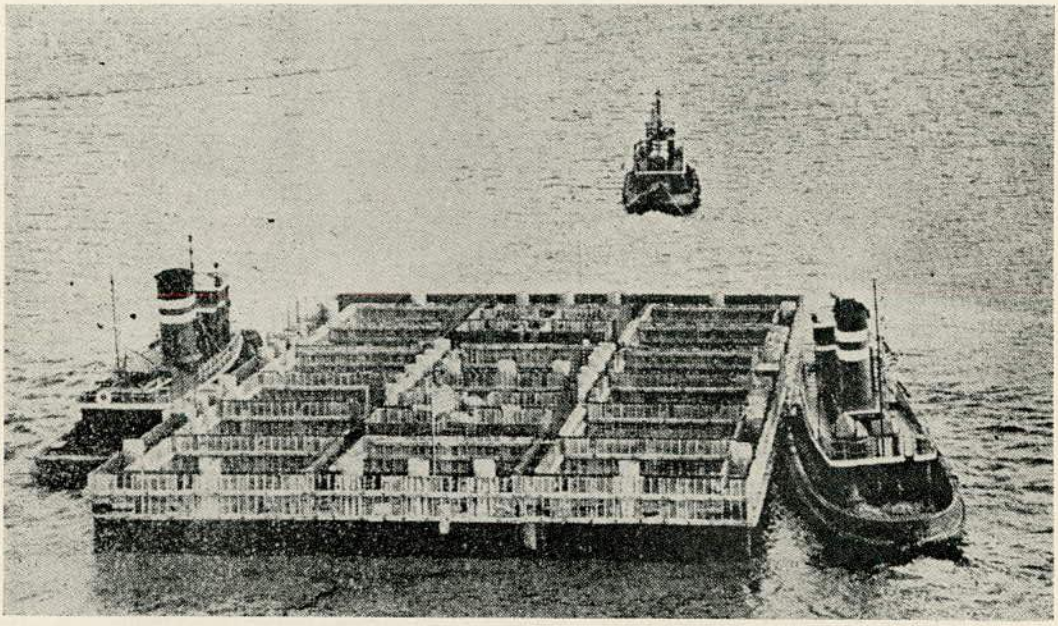


FIGURE 16
Foundation caisson en route to site.

at a depth of about 100 ft along the shore and about 400 ft at the offshore end of the pier. The stratum immediately below harbor bottom is composed of soft to medium stiff Hudson River silt; it varies from 60-ft thickness at the shoreline to 200-ft thickness at the offshore end of the pier.

To carry a new, heavier pier on a convential pile foundation, exceedingly long piles, particularly at the offshore end, would be required. These piles would have to penetrate through the silt and some distance into firm sand. The new piles would have to be located between the many timber piles or

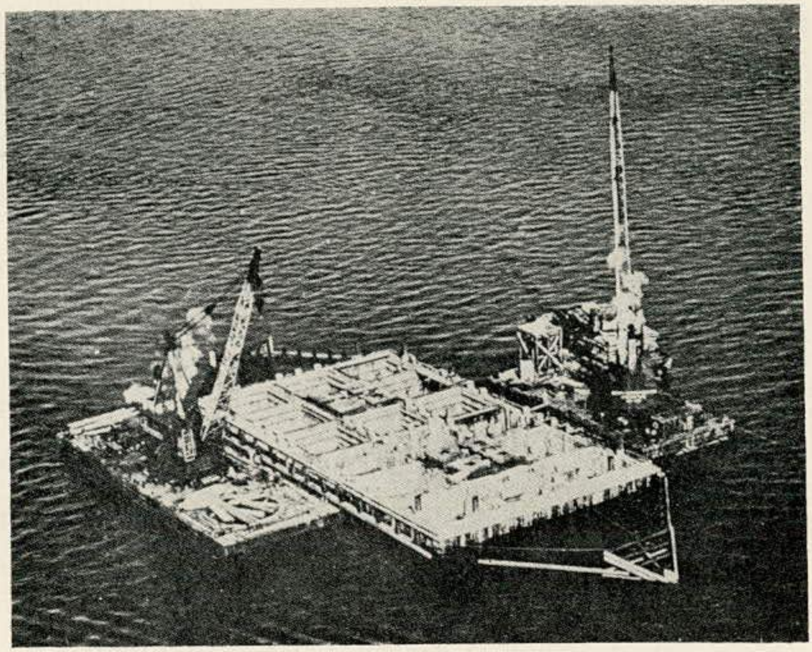


FIGURE 17
Foundation caisson held in place by clusters of timber piles.

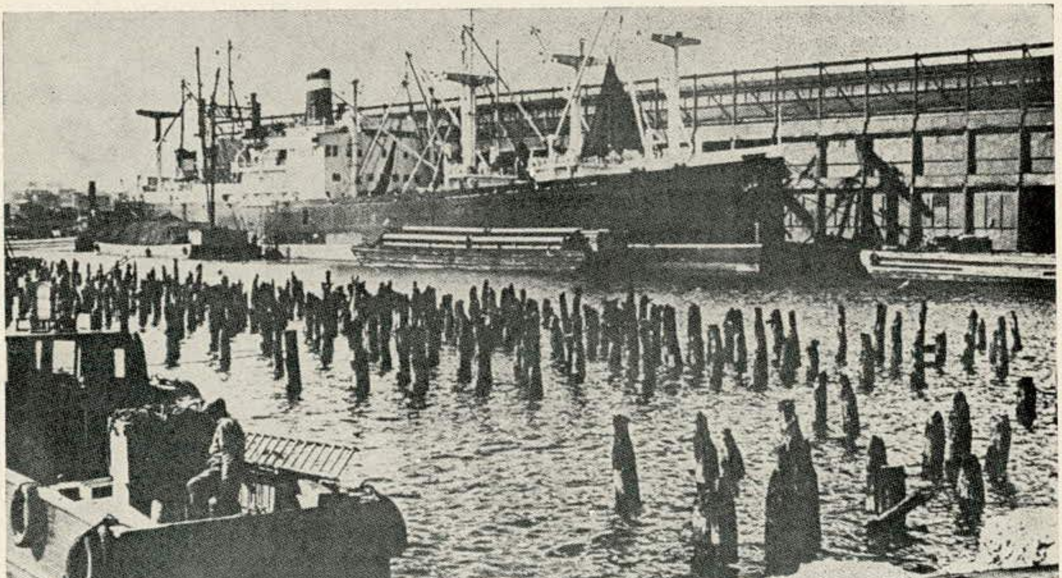


FIGURE 18
Site of Pier 57, New York Harbor, after 1947 fire.

the timber piles would have to be pulled. The timber piles could not be used in conjunction with the new piles due to the difference in stiffness between the silt and the underlying firm material supporting the two different types of piles.

It was decided that by using a floating caisson-type foundation the need for long expensive piles for the finger portion would be eliminated; the existing timber piles could be utilized to carry a load proven by their past performance and the difference

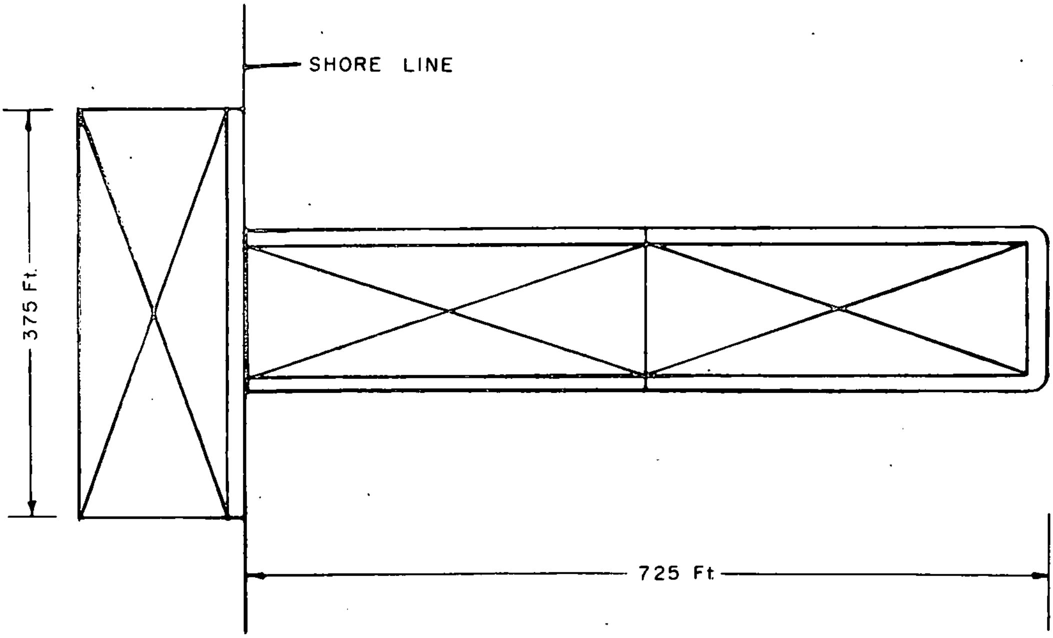


FIGURE 19
Pier 57 plan.

between the capacity of the wooden piles and the load imposed by the new pier would be carried by buoyance of the caissons.

Three caissons were cast, two to form the finger and one for the portion along the shore. The caissons were cast in the basin at Haverstraw in the same manner as described for the Nyack-Tarrytown bridge. The caissons were much larger, however, than any used for the bridge, being 350 ft long by 82 ft wide and 34 ft deep. For the caisson along the shore, piles were driven through the holes cast in the caisson walls to bear on bedrock. For the two caissons forming the finger, the piles driven through the holes in the caisson walls did not penetrate through the silt. These piles are considered to pin the caissons to the foundation.

Before the caissons were brought to the site, the silt and debris between the existing timber piles was removed by clamshell bucket. When the excavation reached about 35 ft below mean low water, the timber piles were cut off. In order to insure that the timber piles would carry their former load without causing settlement of the silt, sand drains were installed between them. The caissons were then brought to the site and the full height of walls and deck constructed. The deck cantilevered out about 35 ft on either side of the caisson. The bottom of the caisson cantilevered out in a similar manner in order to engage more of the existing timber piles.

The caissons were then sunk to rest on the piles by filling them with water. The filling was adjusted to impose full design load on the old piles. After allowing seven months for the foundation to adjust under this loading whatever voids existed between the bottom of the caisson and the foundation were grouted. As the superstructure was erected, water was pumped out of the caissons to maintain the proposed design load on the timber piles.

The finished pier consists of a ground level plus two upper floors as shown *(Fig. 20)*. Also, the roof was designed for storage of bulky cargo, such as automobiles. Further, the space below ground level in the hollow caisson is used for storage. The caissons are divided into various compartments by the walls and elevator access is provided

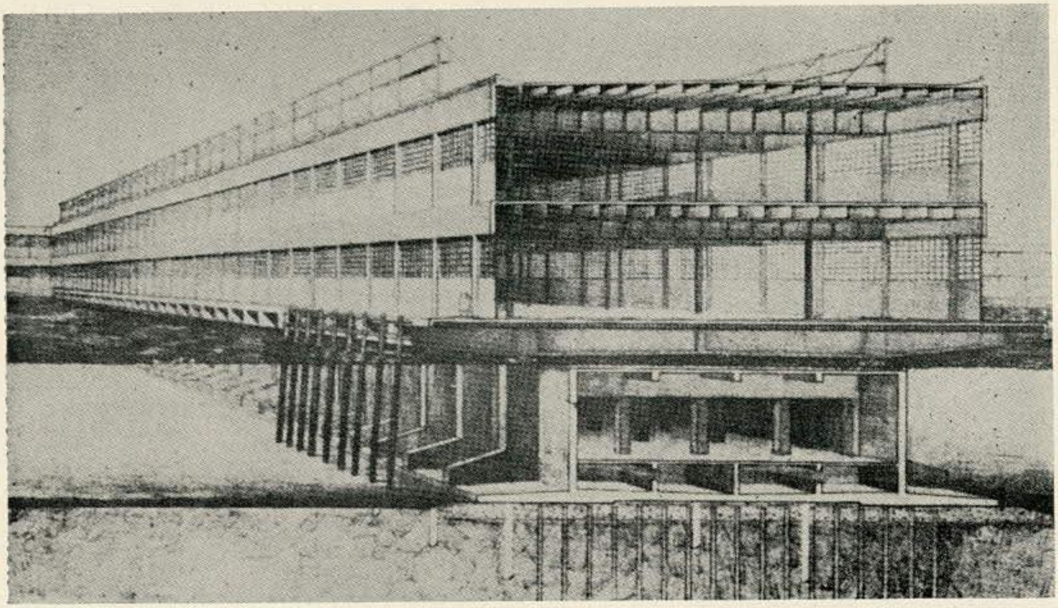


FIGURE 20

New Pier 57 is a complete departure from the conventional. Built of concrete and steel, it will not burn; marine borers will not touch it, and maintenance will be minor. Moreover, its foundation floats, its soil base will be sand-drain stabilized, and its main floor will be framed in prestressed concrete.

to the different compartments. The result is an efficient, high capacity, modern pier constructed on poor foundation materials for about the same amount of money as would be required for a similar pier on reasonably good foundation materials.

Steel Piles with Attachments

In recent years the author has been connected with the design of several projects on which it has been found economical to put attachments near the foot of steel piles in order to increase their bearing capacity. This paper describes the design of attachments for two such projects.

ORE TERMINAL, MOBILE, ALABAMA

In 1953 the Tennessee Coal and Iron Division of the United States Steel Corporation constructed an ore terminal at Mobile, Alabama, for the transshipment of iron ore from ships to railroad cars. A major feature of the terminal was a marginal wharf 1,000 ft long and 80 ft wide. A plan and cross-section of this wharf is shown *(Fig. 21)*.

The owners of the project, being a steel company, desired to use steel H-piles and steel box piles for the pier if at all feasible. Borings made at the site indicated that the subsurface conditions consisted essentially of a deep deposit of sand whose density varied in a consistent pattern with depth. For instance, from about Elevation —40 ft to —85 ft the sand was found to be medium dense, whereas below it, from Elevation —85 ft to —100 ft there exists stiff clay and below 100 ft the sand becomes dense.

Upon driving the test piles for the job, it was found that the simple H-pile and the box pile penetrated through the upper medium dense layer and into the lower dense layer requiring a rather low number of blows per foot for driving. Load tests performed on these piles confirmed that the

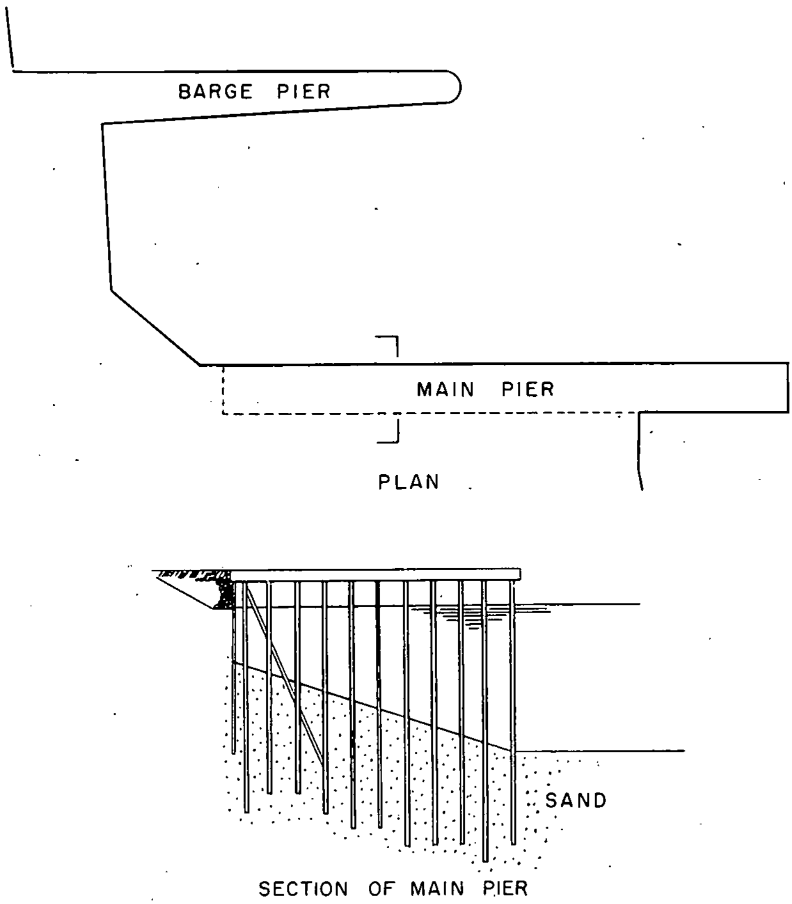


FIGURE 21
Ore terminal, Mobile, Alabama.

bearing capacity was inadequate. The driving records for two typical piles are shown *(Fig. 22)*. During a load test on the steel box pile a 1-in. movement was observed with 138 tons applied to the test pile.

In order to develop the bearing capacity of the piles within a reasonable depth, S. S. Cooke-Yarborough, who was working for the author on this project, suggested that the cut-offs from the H-piles be cut on a diagonal and be welded to the flanges of the H-piles as attachments to increase the point bearing capacity of these piles. The details of the attachment are shown *(Fig. 23)*.

When this was done it was found that piles with attachments at about 6 ft from the bottom of the pile developed high driving resistance in the upper medium dense sand layer. Load tests on these piles confirmed that their capacity was entirely adequate for the proposed design loads which had been based on 10,000 lbs psi, the allowable compressive stress for steel.

Somewhat similar attachments were developed for the box piles. Shown *(Fig. 24)* are typical driving records of piles with such attachments. Superimposed on the above driving records of piles with attachments are the driving records of similar piles without attachments. The marked increase in driving resistance developed by the attachments is evident. For comparison a 1-in. movement was observed when a 250-ton load was applied to the box pile with attachment.

The experience gained from this particular job is that although H-piles are not suitable as displacement piles and are not well

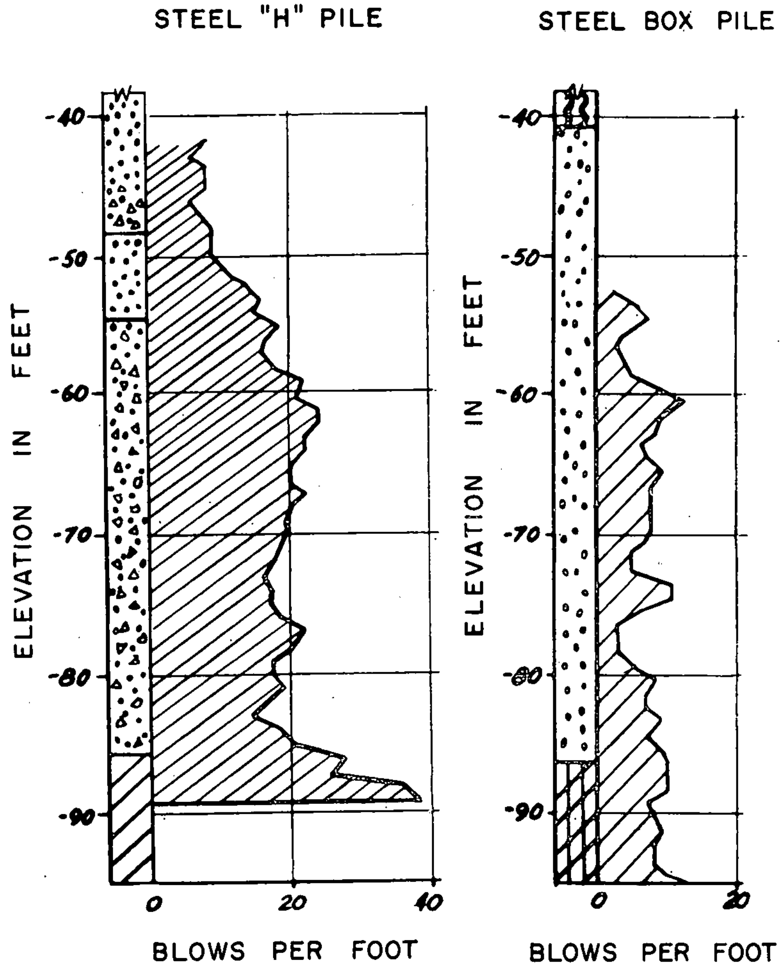


FIGURE 22
Pile driving graphs for steel H and box piles.

suited for developing point bearing capacity in soils, nevertheless, by means of simple, inexpensive attachments applied near the foot of the pile, their point bearing capacity can be greatly increased so that their full structural load capacity can be developed with minimum lengths.

MARINE TERMINAL FOR THE CITY OF ANCHORAGE, ALASKA

At the present time (Sept. 1959) construction is under way on the first deep water pier for the City of Anchorage, Alaska *(Fig. 25)*. Both the soil conditions and the harbor conditions present unusual problems for the design of the piles for the pier. Soil conditions at the site were investigated by a detailed program of undisturbed sample borings. The findings of these borings indicated the general soil profile shown in Figure 25.

Offshore, where the pier is to be located, there is a shallow surficial layer of loose silt and sand. Under this is a 20-ft to 40-ft thick layer of dense silt and sand which is somewhat stratified and below this is a deep layer of stiff clay.

Tide conditions at the site are severe, there being a total variation of about 40 ft from extreme low water to extreme high water and an average daily variation of 30 ft.

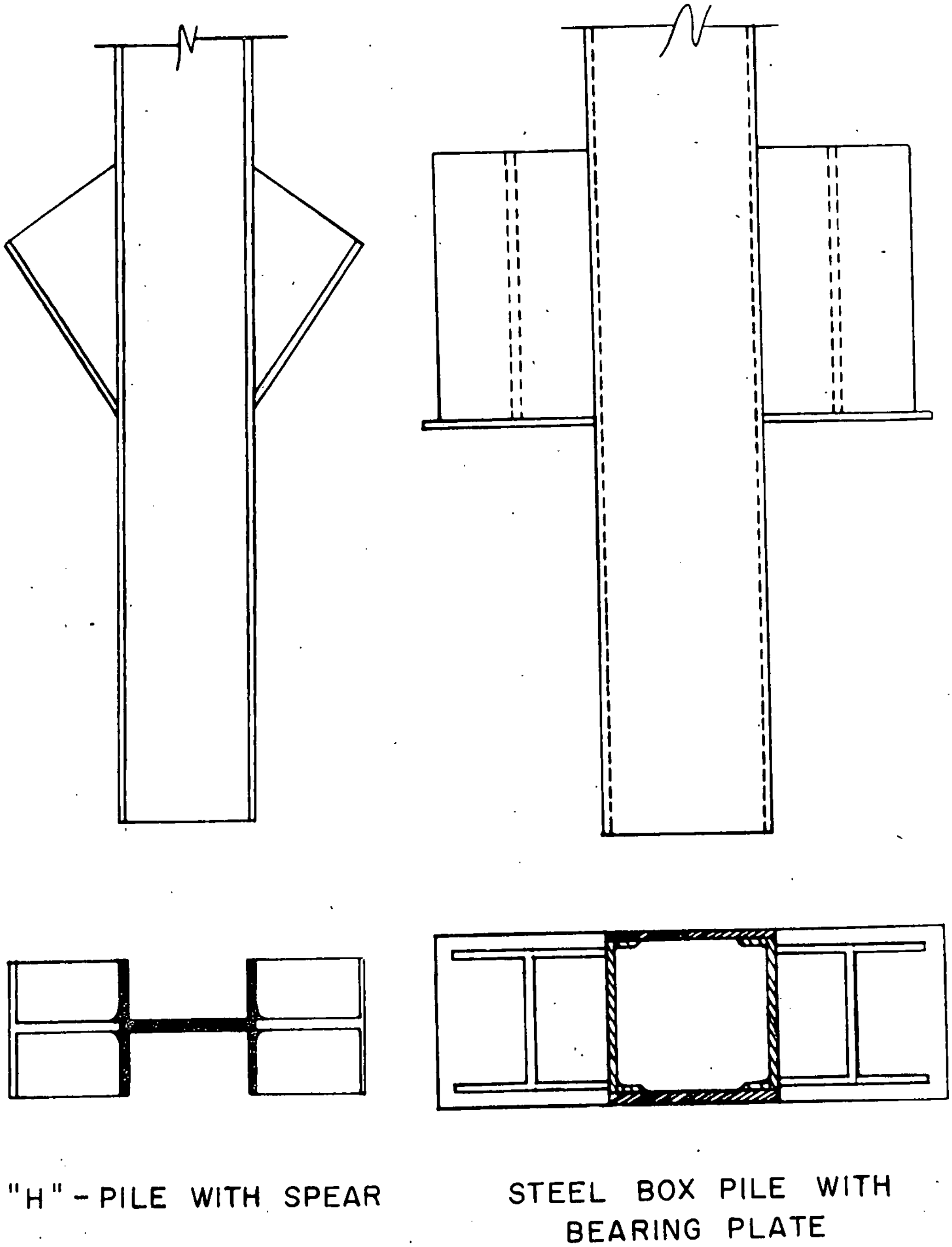


FIGURE 23
Modified steel H and box piles.

In the winter ice floes choke the harbor; the ice shifts both with the tide and wind. The pier must be sufficiently strong to resist the impact of large accumulations of this ice. Also, it is expected that during the winter ice will form on the piles in a manner similar to that which has been observed on adjacent small piers and the area under-

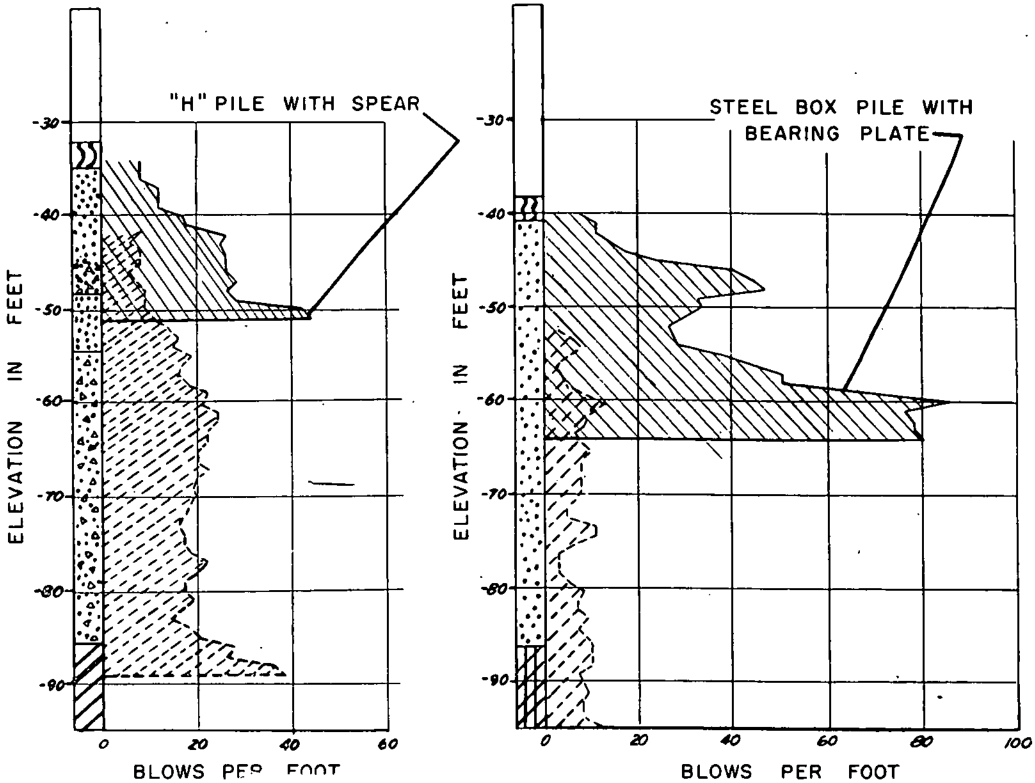


FIGURE 24
Pile driving graphs for modified steel H and box piles.

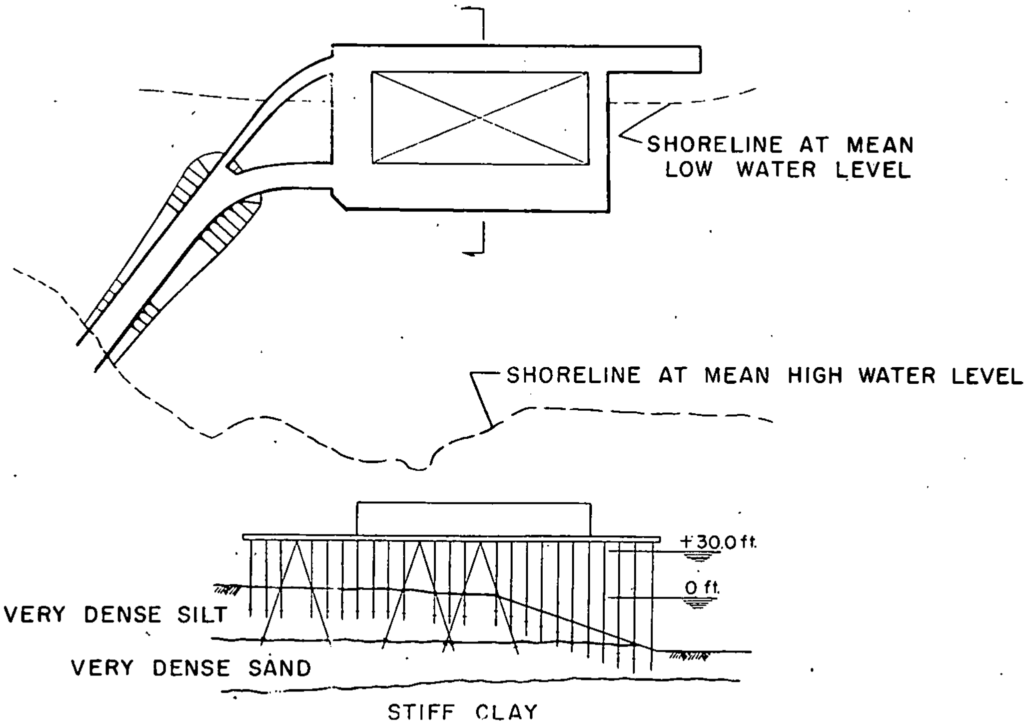


FIGURE 25
Marine Terminal, Anchorage, Alaska.

neath the deck of the pier may become more or less a solid mass of porous ice.

A cellular type of pier construction was considered but rejected because of stability and settlement considerations. The design chosen consists of various size steel pipe piles. At the offshore face of the pier is a row of 3.5-ft diameter caisson type piles. Behind this are four rows of 24-in. diameter and four rows of 20-in. pipe piles and at the inshore area are 17 rows of 16-in. piles.

The cost of shipment of steel to Anchorage is expensive so that the length of piles had to be kept to a minimum. In order to do this, it was necessary that the piles develop their bearing capacity in the stratified silt and sand layer overlying the clay. If the piles penetrated through this layer into the clay, the length of piles would have to be appreciably greater to develop the design bearing capacity. The design provided that attachments be placed on the sides of the piles in sizes necessary to develop the capacity of the piles in the stratified silt and sand layer. The attachments consisted of annular steel plates which were threaded on the pipe piles, welded to the walls of the pipe and stiffened by brackets as shown *(Fig. 26)*.

The steel cut out to form the hole in the center of the annular ring was used as a cover plate at the bottom of the pipe. The diameter of the annular rings had to be determined very carefully by testing in the field. Although it was desired they be large enough to develop adequately the bearing capacity of the piles, they could not be so large as to prevent pile penetration to the minimum embedment required for fixity.

To date approximately half of the number of piles required for the pier have been driven and they have been meeting both the minimum and maximum penetration requirements as well as the criteria of driving resistance to develop their bearing capacity. It has been found necessary to make only one small change in size of plate to adjust the piles to variations in conditions encountered so far at the site. The correlation between driving resistance and static load bearing capacity was developed by a series of pile load tests performed at the site.

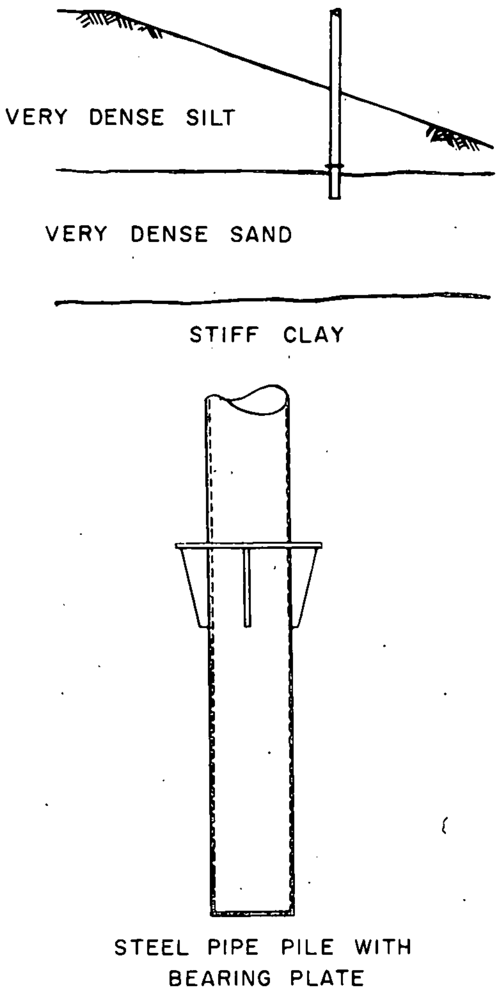


FIGURE 26
Modified steel pipe pile at Anchorage Marine Terminal.

A very large saving in cost of the project was effected by designing the piles to develop their bearing capacity in the stratified silt and sand layer overlying the deep clay deposit.

Current Practice in Soil Sampling in the United States

John Lowe, III

This paper presents a summary of the principal types of soil samplers in common use in the United States. Each type of sampler is described and its particular applications to sampling indicated.

The one-inch retractable plug sampler developed by O. J. Porter about 1930 is shown *(Fig. 1)*. The sampler consists of an inner retractable plug and piston rod, an outer casing and master tube with brass liners, and a driving mechanism.

The steps in the operation of the sampler are as follows. With the piston and plug locked to the casing *(Fig. 1A)* the sampler is driven into the ground by lifting the 30-lb drive weight by hand and letting it fall upon the drive head.

When the sampler reaches a depth at which sampling is desired, the piston rod is rotated to unlock it from the casing, raised a distance of 2 ft, and locked to the casing again *(Fig. 1B)*. The unit is then driven 2 ft deeper to obtain a sample *(Fig. 1C)*. At the end of sampling *(Fig. 1D)* the plug is rotated further to develop a tight seal at the top of the sampler. The entire unit is then jacked out of the ground *(Fig. 1E)*.

The next sample is taken by driving the unit down the same hole, but to greater depth. Disturbed but representative samples are obtained. The sampler is used primarily for determining the thickness of surficial deposits of soft silts and clays as occur in swamps and estuaries. The unit can be operated to depths of 60 ft or more. One of its main advantages is that it can be hand-carried into swampy areas difficult of access by conventional equipment. The unit is used both for reconnaissance investigations and for mapping soft strata between more elaborate sample borings.

The split barrel sampler is the most commonly used sampler. As illustrated *(Figs. 2, 3)*, it consists of a barrel shoe, a split barrel, a solid sleeve, and a sampler head. When the shoe and sleeve are unscrewed from the

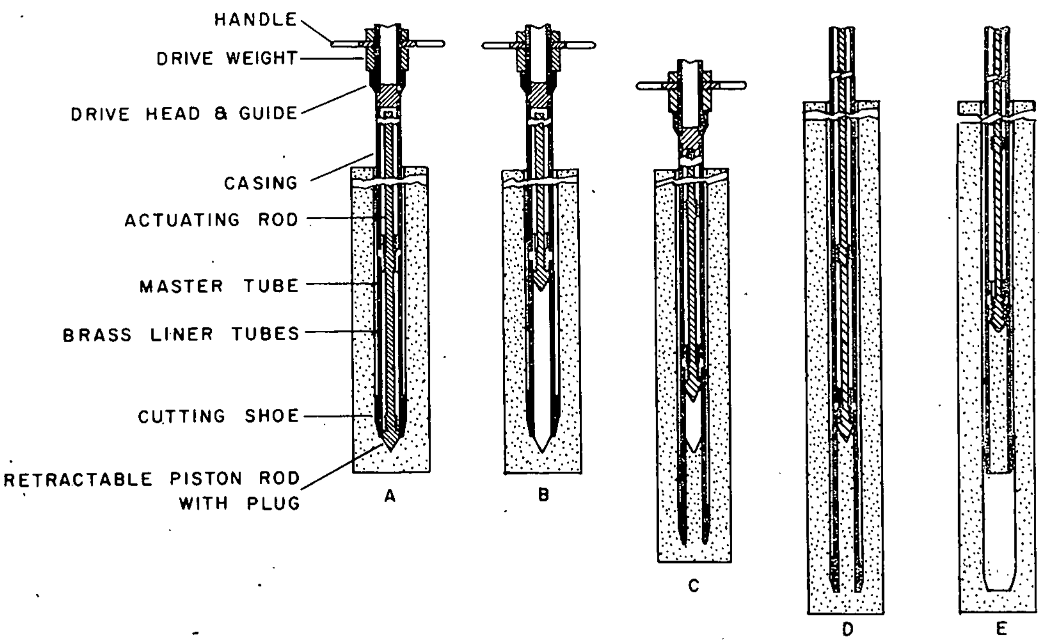
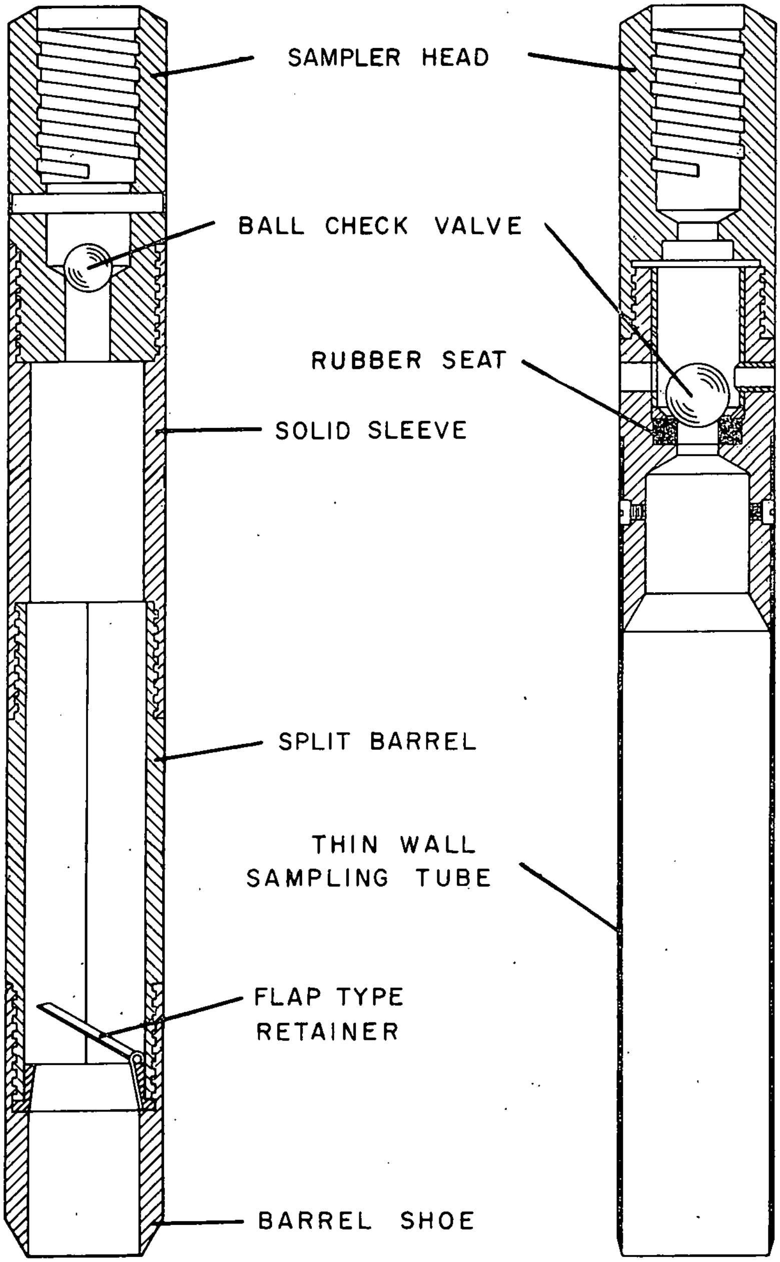


FIGURE 1
One-inch retractable plug sampler.



FIGURES 2 and 3
Split barrel sampler and thin wall "Shelby Tube" sampler.

split barrel, the two halves of the barrel may be separated and the sample easily extracted.

To facilitate recovery of samples of cohesionless type materials, a ball valve is incorporated in the head of the sampler. This valve seats when the sampler is being withdrawn from the soil and if the sample tends to slide out, a vacuum created at the top of the sample helps to retain it. At the lower end of the sampler a flap type retainer may be used. This permits soil to enter during sampling but upon withdrawal of the sampler from the ground the flap closes and retains the sample.

An alternative type of retainer consists of a crown-shaped spring. The points of the crown deflect toward the center of the sampler. As soil enters the sampler it easily pushes aside the points. If upon withdrawal of the sampler from the ground the sample tends to slide out, the points of the spring move inward and grip the sample. Generally, the sampler has an inside diameter of 1½ in., an outside diameter of 2 in., and a length of 18 to 24 in. For sampling soils which contain gravel sizes the inside diameter may be increased to 2½ in. or more. The wall thickness in all cases is about ¼ in.

Sampling is accomplished by driving the sampler into the ground with a drive hammer, the weight of which depends upon the size of the sampler. For the 1½ in. inside diameter sampler, a weight of 140 lbs is generally used; for the larger samplers larger hammers are used.

It is common practice during sampling operations to record the number of blows required for each 12 in. of penetration of the sampler; many engineers record the number of blows for each 6 in. of the 12 or 24 in. total penetration.

The number of blows required to drive the 1½ in. sampler a distance of 12 in. with a 140-lb hammer falling a distance of 30 in. is the standard penetration resistance which has been developed by Terzaghi and Peck. The correlations between standard penetration resistance and stiffness of clays and density of sands developed by Terzaghi and Peck are widely used, probably with more faith in their accuracy than can be justified.

Practically all subsurface exploration programs are initiated with split barrel sampling. Frequently foundation conditions are such that design can be made on the basis of the split barrel sample data. This is particularly the case where a minor type structure is involved. In other instances the split barrel sample borings are supplemented with sampling by one or more of the undisturbed type samplers described below.

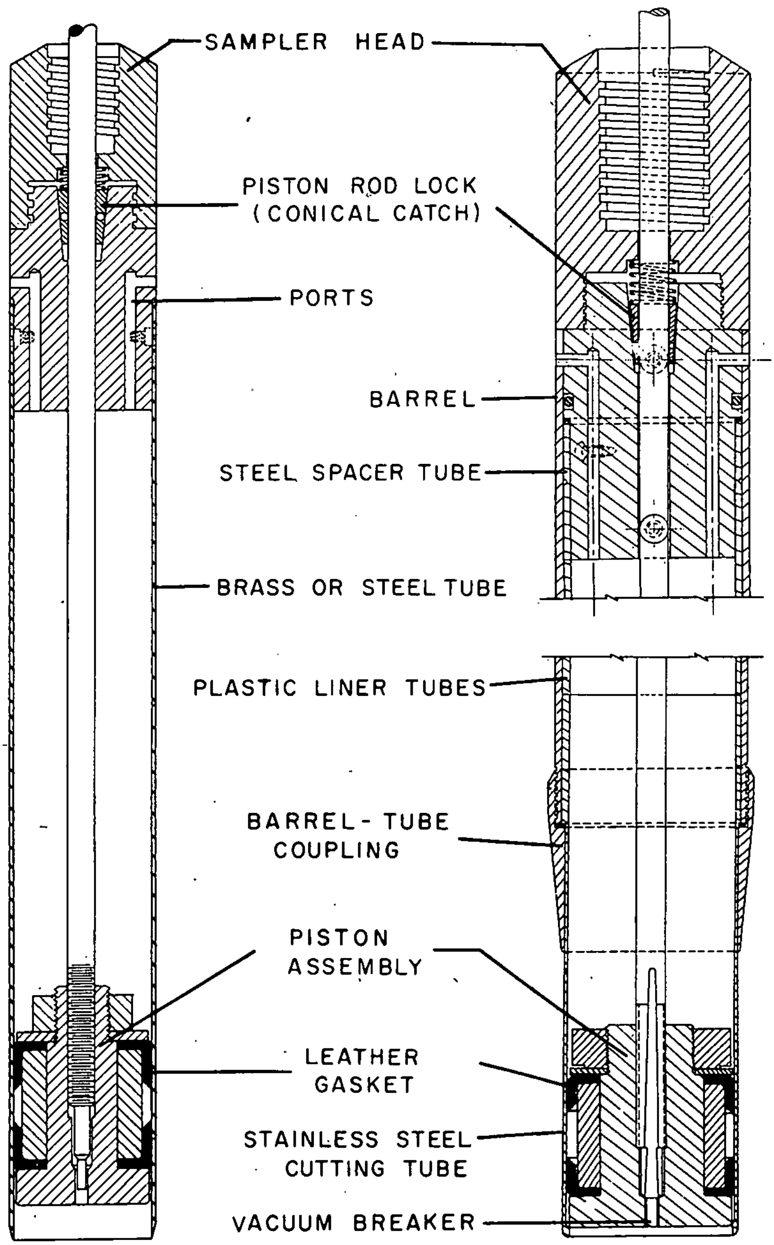
The thin-wall tube sampler (Shelby tube sampler) is the next most commonly used sampler in the United States. This sampler is illustrated in Figure 3. It consists of a thin-wall steel or brass tube and a head section. The head section is similar to that used for the split barrel sampler. The thin-wall tube is drawn in at the lower end and reamed so that the inside diameter of the cutting edge is ½ to 1 per cent less than that of the tube. The most common sizes of tube are 2-in. and 3-in. diameters. However, occasionally samples are taken with tubes 4 in. and 6 in. in diameter. The length of the sampler is generally about 30 in. Some engineers coat the inside of the tube with lacquer to minimize corrosion effects on the sample. Ports are provided in the head to permit easy escape of drilling fluid from the sample tube as the sample enters it.

The sampler is used primarily for sampling soft to stiff cohesive soils. For such soils, the wall thickness of the tube is about $\frac{1}{16}$ in. and the sampler is pushed into the soil at a rate of ½ to 1 ft per second.

At times the sampler is used for obtaining undisturbed samples of hard cohesive soils or sand. In such instances the wall thickness is about ⅛ in. and the sampler is driven into the soil. After the sample has been taken and the thin-wall tube has been removed from the head, the ends of the sample may be coated with petro wax or paraffin, covered with metal caps and shipped to the laboratory.

At the laboratory the sample tube is cut into 6- to 9-in. lengths by means of a saw or a thin abrasive wheel, and the sample is ejected from the sections of tube. Occasionally the sample is ejected in the field, cut into 6- to 9-in. lengths and placed in cylindrical waxed containers somewhat larger than the sample. The space between the sample and container is filled with petro wax or paraffin. Samples from 2-in. diameter tubes are suitable for unconfined compression and triaxial compression tests. Samples from no less than 2¾-in. diameter tubes are required for consolidation tests.

The thin-wall stationary piston sampler is the thin-wall tube sampler plus a piston and piston rod. The sampler is illustrated *(Figs. 4, 5)*. The piston rod is ½ in.



FIGURES 4 and 5
Thin wall stationary sampler and thin wall stationary piston sampler with liners.

in diameter and fits easily inside the hollow drill rod. Joints in the piston rod are displaced about 6 in. from joints in the drill rods.

The unit may be lowered on the drill rod since a conical ball-bearing catch, termed the piston rod lock on the figure, prevents the piston rod from slipping downward with respect to the head of the sampler. To prevent upward movement of the piston as the sampler is lowered into the bore hole, the piston rod has a short section of left-handed threads which engages a matching section of threads in the sampler head.

By rotating the piston rod counter-clockwise, the rod is threaded into the sampler head and the piston is locked at the bottom of the sampler; to release the piston from the sampler, the piston rod is given several clockwise turns. This method of locking the piston during the lowering of the sampler into the bore hole is required for sampling deep below ground water or in bore holes filled with heavy drilling fluid in order to prevent the piston from rising under the fluid pressure.

The procedure used to obtain a sample is as follows. The sampler, with its piston located at the base of the sampling tube, is lowered into the bore hole. When the sampler reaches the bottom of the hole, the piston rod is held fixed relative to the ground surface and the thin-wall tube is pushed into the soil. The fixed piston tends to prevent excess soil from entering the thin-wall tube at the beginning of sampling and too little soil from entering near the end of sampling. It also acts more positively to retain the sample than the ball valve of the thin-wall tube sampler. After sampling, the vacuum between the piston and the top of the sample is broken, using a device provided for this purpose in the piston; the piston is then removed from the sampler.

The stationary piston sampler is used for sampling soft to stiff cohesive soils. It is jacked into the soil at a rate of ½ to 1 ft per second; it is never driven. The most frequently used size is 2.8 in. inside diameter.

A thin-wall stationary piston sampler with liners, developed by the author together with the Acker Drill Company, is shown in Figure 5. The piston and piston rod, the lower 6 in. of the sampler and the head section, are all very much the same as in the standard stationary piston sampler. The lower tube is made of stainless steel, and is connected to a barrel containing liners. Originally, the liner tubes were made of brass but now are made of laminated phenolic resin-impregnated paper.

The liners are 6 in. and 9 in. in length and 0.1 in. in wall thickness. They are inert chemically and electrolytically; therefore samples can be stored in them for long periods of time without corrosion effects. The liners are watertight and tough. Because of their short lengths, the liner tubes do not have to be cut in the laboratory and disturbance to the sample from this operation is eliminated. The liners are reusable and light in weight for shipment.

Because the lower portion of the sampler consists of a 6-in. long thin-wall tube and a 3-in. long tapered coupling, the effect of the thicker walled barrel section on sampling is believed to be negligible. The sampler is used to sample soft to stiff cohesive soils. However, because of the high penetration resistance of the barrel section the sampler requires greater thrust than the standard thin-wall piston sampler when used in sands and very stiff cohesive soils. The sampler with plastic liners has been in use more than two years and has proved entirely satisfactory.

In the author's laboratory the sample is ejected from the liner as illustrated *(Fig. 6)*. A close fitting piston is brought in contact with the lower end of the sample. As the piston is jacked upward the liner is held stationary by the restraining plate. The sample is ejected from the liner and into a specimen cutting punch which in effect is a miniature thin-wall tube sampler. Excess material is trimmed away as the sample moves into the punch.

Jacking is continued until soil reaches the top of the punch. Then the ends of the specimen are trimmed by sliding a wire across the ends of the punch. Alternatively, the full 2.8-in. diameter sample is used; this is frequently the case in triaxial tests. Other laboratories generally use turntables and wire saws for trimming less than full diameter specimens from ejected samples.

Samplers somewhat similar to the thin-wall stationary piston sampler with liners have been developed by the firm of Dames & Moore and by Prof. Housel at the University of Michigan. Generally, 1-in. brass rings are used as liners in the latter two samplers.

The Swedish foil sampler has been introduced into the United States in the last two years. Sprague & Henwood, Inc. have the franchise for its use in the eastern part of the country and International Engineering

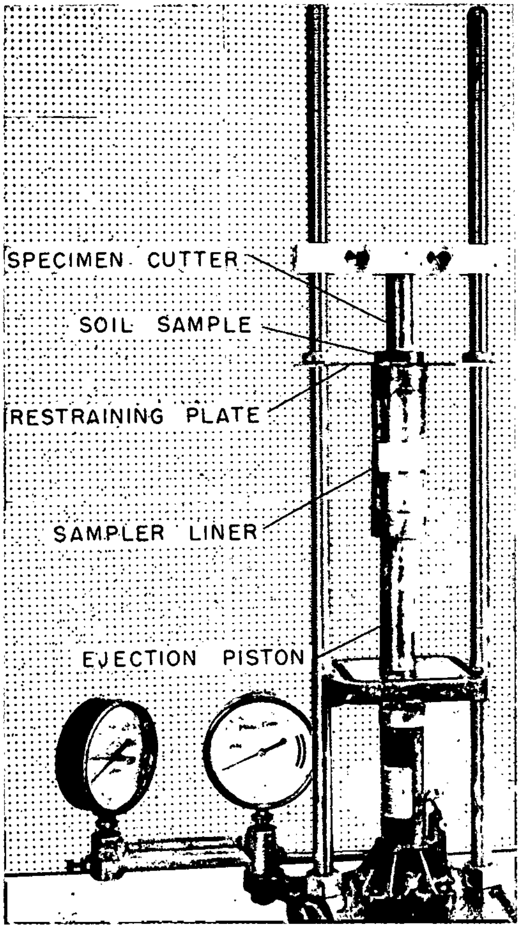


FIGURE 6
Sample ejector and specimen cutter.

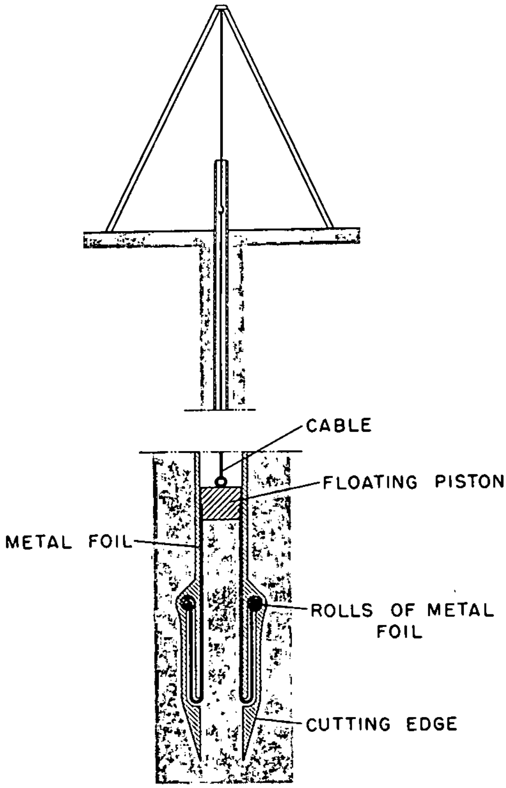


FIGURE 7
Schematic drawing, principle of Swedish foil sampler.

in the western. The basic principle of the sampler is illustrated *(Fig. 7)*.

The sampler consists of a sampling tube containing rolls of metal foil and a more or less fixed piston to which the ends of the foil strips are attached. As the sampling tube is pressed into the ground the strips of foil unwind and encase the sample. Thus, friction between the sample and the inside of the sampling tube is eliminated. Friction does develop, however, between the foils and the inside of the sampling tube and the tensile strength of the foils must be adequate to carry the friction accumulated over the extended length of the foils.

Longitudinal and transverse sections of the sampler are shown *(Fig. 8)*. They illustrate how 16 rolls of foil are incorporated into the sampler and are arranged to completely encase the sample.

Because of the elimination of friction, continuous samples up to 75 ft long may be taken. The sampler is uniquely adapted for sampling of thinly stratified soils and for sampling of very soft and semiliquid soils which frequently occur at harbor bottoms. By sampling through the very soft surficial layers and into somewhat stiffer layers below, enough friction can be developed between the foil and the stiffer soil to hold both the lower layers and the overlying very soft layers in the sampler during withdrawal from the ground. It is anticipated that use of this sampler will increase in the United States.

The Osterberg piston sampler is a recent modification of the thin-wall stationary piston sampler. The operation of the sampler is illustrated *(Fig. 9)*. The new feature is a second piston, termed the actuating

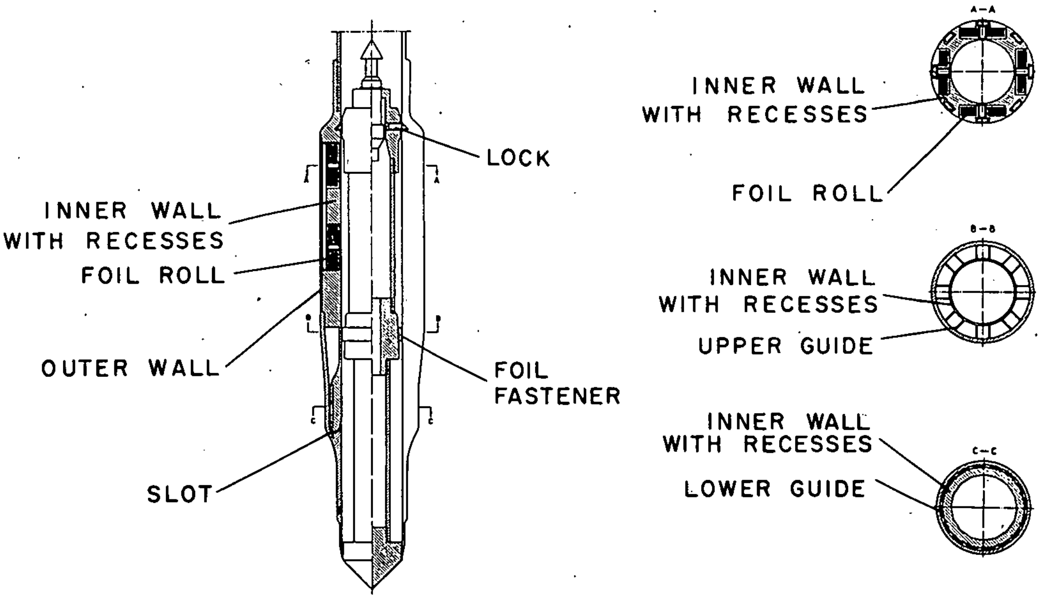


FIGURE 8
Swedish foil sampler head.

piston, and a pressure cylinder. To take a sample, fluid pressure is brought to the top side of the actuating piston. The pressure forces the actuating piston and the sampling tube to move down. At the end of sampling the actuating piston comes in contact with the stationary piston; thus the sampler cannot be over-pushed.

Another advantage of the sampler is the fact that only one set of drill rods is required. These rods are used for holding the stationary piston. The actuating rods used in the standard stationary piston sampler are eliminated by the use of the actuating piston. Pressure to operate the actuating piston is brought from ground surface to the sampler through the hollow drill rods. To date the sampler has been used primarily in the 5-in. diameter size to procure samples of clay for consolidation tests.

A variation of this sampler has been developed by McClellan. In the McClellan

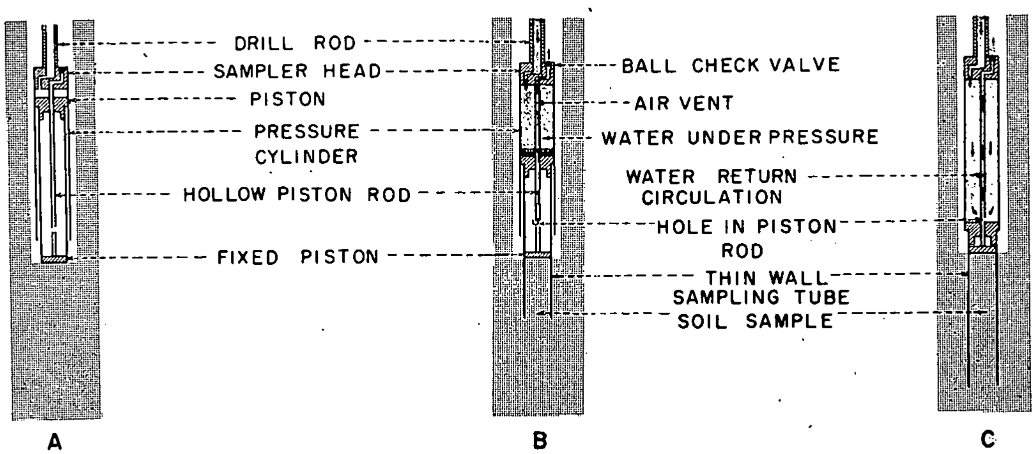


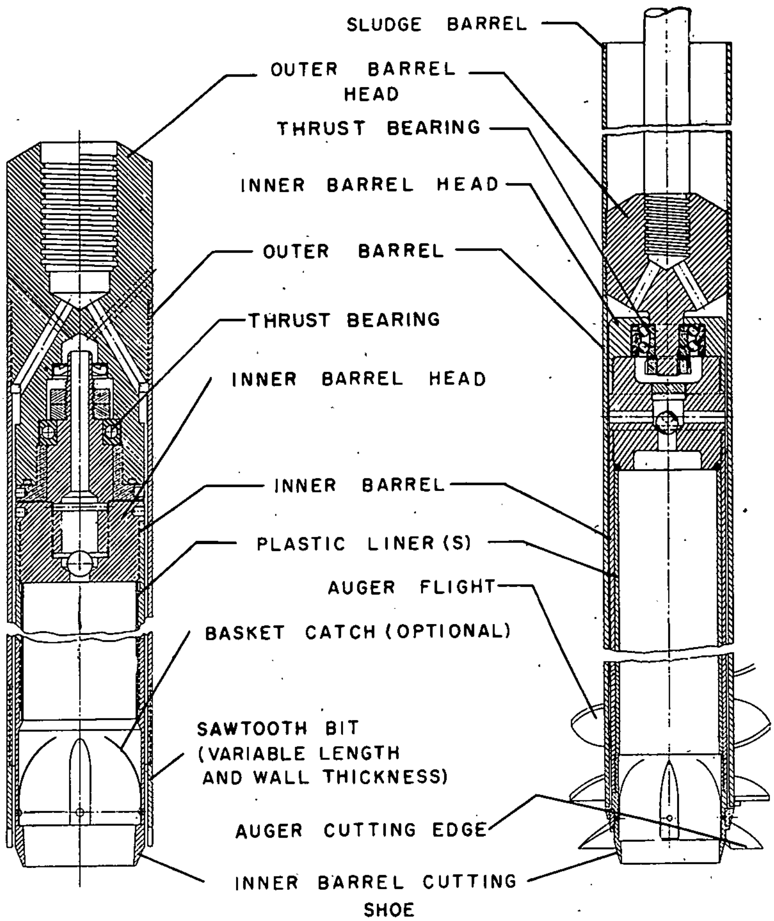
FIGURE 9
Osterberg piston sampler.

variation the actuating piston is held in position with a shear pin while pressure is built up above it. When sufficient pressure has been developed, the pin shears and the sampler moves rapidly into the soil. The purpose of this variation is to permit a quick jacking of the sampler into the soil.

The Denison double tube core barrel soil sampler *(Fig. 10)* was developed by H. L. Johnson about 1939 for the Denison Dam project in Texas.

The sampler is different in operation from the samplers previously described. It consists of an outer rotating core barrel with bit and an inner non-rotating sample barrel with liners and cutting edge. Either a carbide insert bit or a hardened steel sawtooth bit may be used depending on the material to be sampled. The cutting edge of the sample barrel may be set 0 to 2 in. ahead of the leading edge of the coring bit, depending on soil conditions. A basket type sample catcher may be inserted behind the cutting edge of the sample barrel to help retain the sample upon withdrawal from the ground. The inner head contains ports for the escape of drilling fluid and a check valve for the closing of the ports while raising the sampler.

In operation the outer core barrel is rotated as the sampler is pressed downward. Drilling fluid, generally drillers' mud, is circulated downward between the two barrels, through and under the bit and then upward between the outer barrel and the sides of the bore hole. The cuttings of the bit are removed by the circulating fluid.



FIGURES 10 and 11
Denison double-tube core barrel soil sampler and Tams double-tube auger sampler with liners.

The sampler was originally constructed for taking 6-in. diameter soil cores and this is still a frequently used size. At the present time the sampler is available also in 2¾- and 4-in. diameter sizes. The author was instrumental in developing the 2¾-in. size which fits inside the standard 4-in. pipe casing often used for borings in the United States. The author's sampler uses a 24-in. long, $\frac{1}{16}$-in. thick liner tube made of phenolic resin-impregnated laminated paper. This tube can be cut easily by saw in the laboratory for removal of the soil core.

The sampler is designed for use in stiff to hard cohesive soils and in sands. By using drillers' mud, a vacuum valve, and a basket catch, samples of clean sands may be recovered. The sampling action is ideal in that the cutting edge of the inner barrel easily displaces soil into the annular space cut by the bit on the outer barrel. In soft soils the cutting edge leads the bit by as much as two in.; in very hard soils the cutting edge is located approximately flush with the bit.

The Tams double tube auger sampler with liners *(Fig. 11)* operates on a principle similar to that of the Denison double tube core barrel soil sampler except that a helical conveyor rather than circulating drilling fluid is used to remove cuttings made by rotation of the outer barrel.

The sampler has a stationary inner barrel with plastic liners and is equipped with a ball valve at the top and a basket catch at the bottom. The inner barrel is forced into the ground without rotation by means of a thrust bearing incorporated in the head of the sampler. The sampler was developed by the author in 1951 for sampling sandy silt above groundwater table. Excellent results were obtained in the determination of the natural unit weight and water content of the material. The sizes of the liners is 2¾ in. inside diameter. To date the sampler has not been used extensively; however, the author feels that it offers good possibilities particularly when constructed in a larger and stronger version.

A Series M double tube core barrel is illustrated *(Fig. 12)*. This core barrel is used for sampling soft rocks which would be eroded by the flow of drilling fluid. The core barrel is similiar to the Denison double tube core barrel except that the inner barrel does not have a cutting edge projecting beyond the bit of the outer barrel nor does it have a liner. Also, a standard rock core lifter is used instead of the basket catch used in the Denison sampler.

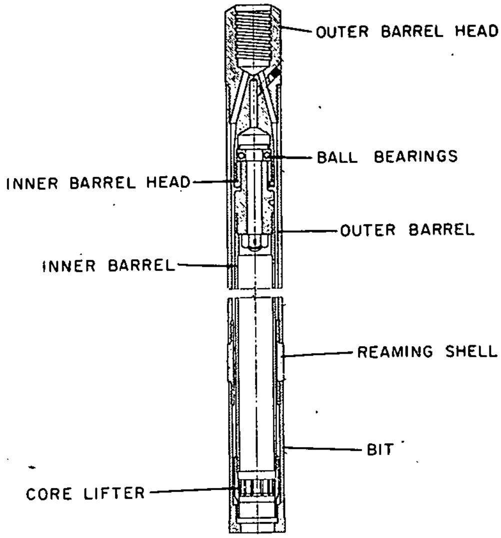


FIGURE 12
Series M double-tube core barrel.

The Vane Shear Apparatus is used extensively in the United States for determining the in-situ shear strength of soft cohesive materials *(Fig. 13)*. Several variations of the apparatus are available. The essential features are as follows. The vane is mounted on a small diameter rod which permits the vane to be pushed below the bottom of the drill hole into undisturbed material. Ball bearings are provided along the drill rod to minimize friction between the drill rod and the casing of the bore hole. A torque wrench which rests on a thrust bearing at the top of the casing is used to measure the torque required to rotate the vane. Shown *(Fig. 14)* is this portion of the apparatus.

Vanes have been developed which can be used in connection with the 1-in. retractable plug sampler. In this case, the sampler with the vane adjacent to the plug is driven into the ground to desired elevation, then the vane is pushed ahead of the plug and the shear test performed. Next the plug sampler

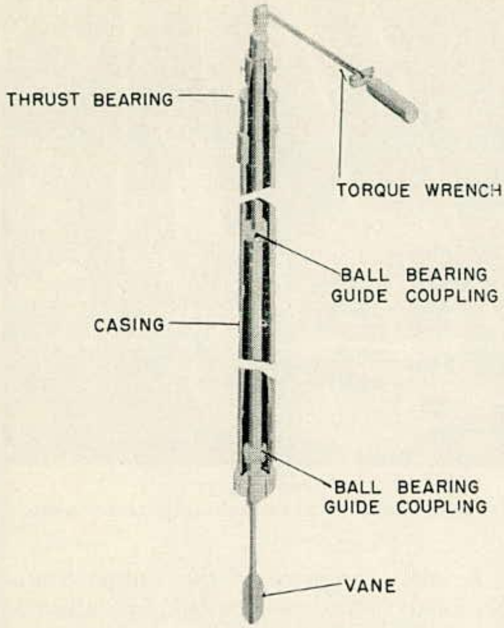


FIGURE 13
Schematic drawing, vane shear apparatus.

is driven down to contact the vane again, and the two are driven as a unit to the next elevation for testing.

Drill rigs used for putting down bore holes and samplers are available in a wide variety of sizes and types. A hand-operated rig, known as the Packsack, is shown *(Fig. 15)*. This is a rotary unit of 5.5 horsepower which weighs only 32 lbs and can be held in position by two men. It is particularly well adapted for reconnaissance work in rough country.

A light rig which is popular for general foundation investigation work is the Acker Teredo. This rig can be operated by its own power unit or by power take-off from a vehicle such as a jeep. The power units generally used consist of diesel engines, gasoline engines and electric motors whose power ratings range from 15 to 30 horsepower. Shown *(Fig. 16)* is a jeep-mounted Teredo operated by power take-off; shown *(Figs. 17, 18)* are trailer-mounted and skid-mounted Teredo rigs with their own power units.

A skid-mounted Sprague & Henwood 40C rig is shown *(Fig. 19)*. This is a 20.2 horsepower unit which is also popular for founda-

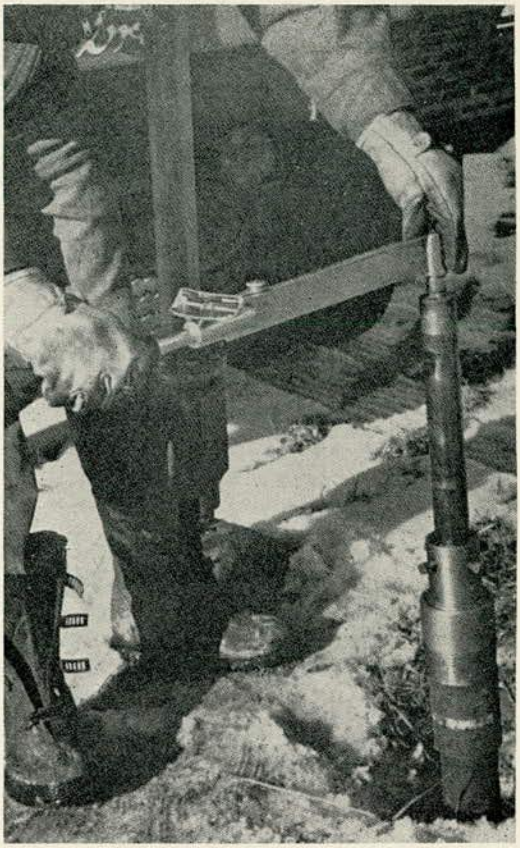


FIGURE 14
Vane shear apparatus torque wrench and thrust bearing.

tion investigaton programs. The hydraulic head is shown in open position in the figure and the gears of the drill head may be seen. A larger Sprague & Henwood rig, the 142, with 30 horsepower is shown *(Fig. 20)*. A still larger rig which has been developed recently is the Acker Presidente shown *(Fig. 21)*. The latter two rigs are used only in large diameter and deep foundation exploration work. Failing rigs comparable in size to the latter two rigs are also quite popular in the United States.

In the previous discussion of samplers and drilling rigs the author has endeavored to include the more commonly used sampling and drilling equipment and to point out the more recent developments in such equipment in the United States. The list cannot help but be influenced by the author's particular viewpoint.

FIGURE 15
Acker Packsack portable diamond core drill.

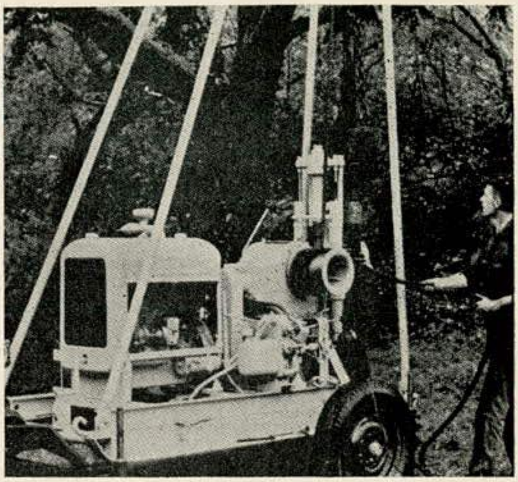


FIGURE 17
Trailer-mounted Acker hydraulic feed Teredo.

A task committee of the United States National Committee for the International Society of Soil Mechanics and Foundation Engineering plans to circulate a questionnaire to all engineers and contractors in-

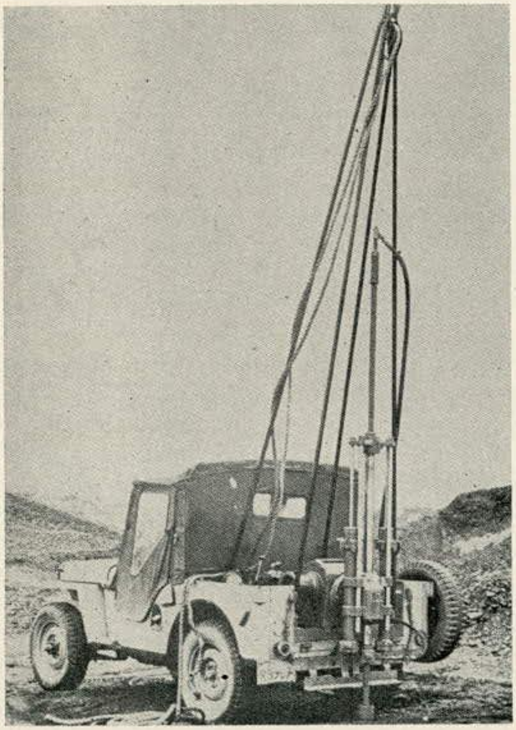


FIGURE 16
Jeep-mounted Acker hydraulic feed Teredo power take-off drive.

FIGURE 18
Skid-mounted Acker hydraulic feed Teredo.

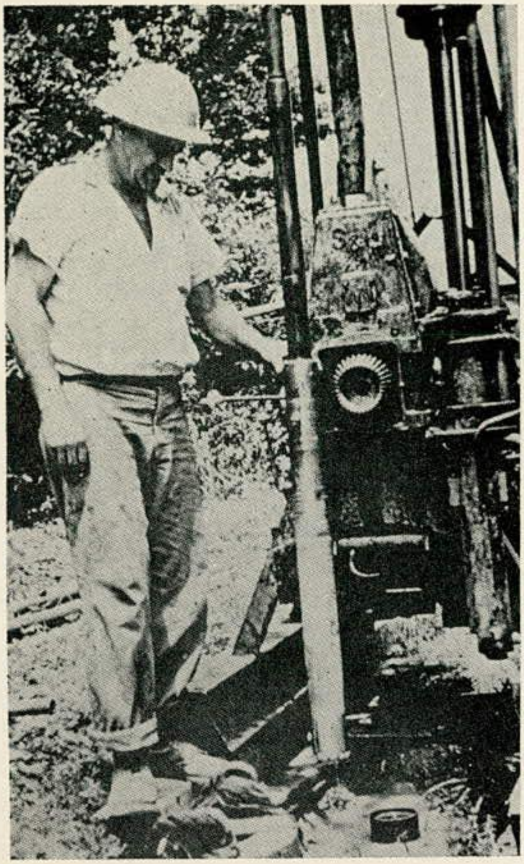


FIGURE 19
Sprague and Henwood 40C hydraulic feed.

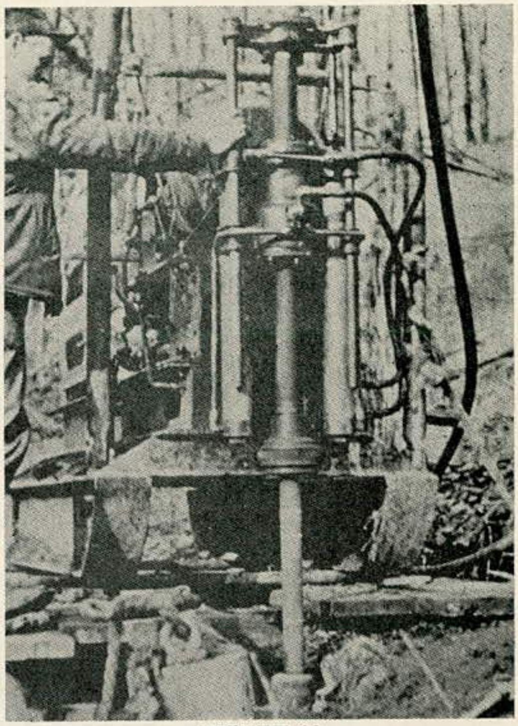


FIGURE 20
Sprague and Henwood 142 hydraulic feed.

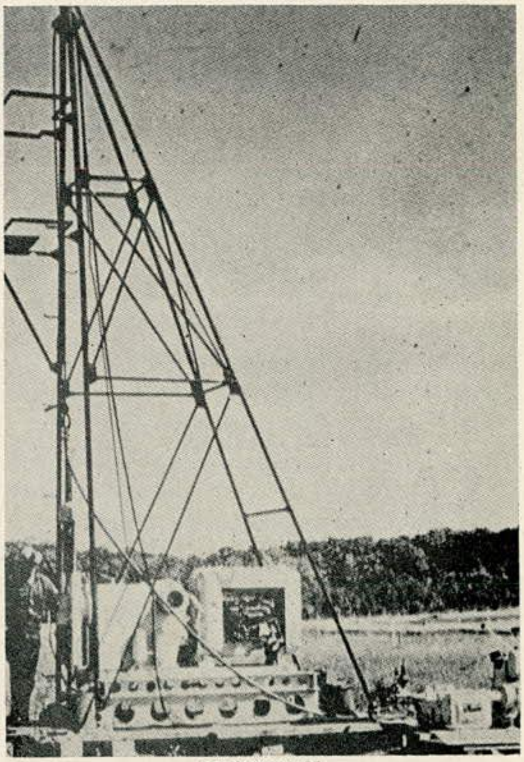


FIGURE 21
Acker Presidente high-capacity hydraulic feed drill.

volved in site investigations in the United States to determine more accurately current practices in sampling. It is hoped that the results of the questionnaire will be available for the next International Conference. The United States National Committee also has a task committee which is part of the International Society's committee on investigation of the standard penetration test.

The author is indebted to the owners of the Acker Drill Company and Sprague & Henwood, Inc., for supplying several of the drawings and photographs.

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Certain Features of Lateral Pressures of Soils

Gregory P. Tschebotarioff, *Professor of Civil Engineering, Princeton University*

In my paper I intend to give a brief review of experimental investigations of two types of retaining structures carried out in recent years in the United States and in England.

In the first part of my report I shall describe special features of lateral pressures on inclined struts of wide open braced cuts. In the second part, I shall outline some aspects of the design of anchored sheet pile bulkheads.

In both cases, the interaction between the retaining structure and the soil is so complex that up to now it has not permitted rigorous mathematical analysis thereof. Observations and measurements on construction sites have invariably preceded even approximate theoretical explanations of corresponding modern methods of design proposed by various investigators. Therefore, both in the first and in the second parts of my paper I shall at first briefly touch upon preceding studies of this kind.

(1) Lateral Pressures of Sandy and of Silty Soils Against Inclined Struts of Sheet Pile Walls

The practical experiences during the construction of the first New York subways during the first decade of our century already showed that the lateral pressures of soils against the upper rows of struts exceeded appreciably the values obtained from design diagrams of the customary triangular shape.

Measurements performed during the following quarter of a century on subway construction sites in New York, Berlin and Munich confirmed that in such cases the actual lateral pressures of sandy soils have a parabolic form *(Fig. 1)*.

On the basis of these measurements, Terzaghi *(Ref. 1)* proposed in 1941 a design diagram which had the form of a trapezoid. This diagram was established in a purely empirical manner as an envelope of pressure diagrams obtained by measurements on various sections of the Berlin subway.

Other studies showed that the increase of the lateral pressures on the upper struts of braced cuts is related to the nature of the deformations of the sheet pile wall as shown diagrammatically on Figure 1.

A rigorous theoretical solution of the problem has not yet been obtained and is greatly hindered by the fact that at different depths in the soil two physically different phenomena, i.e., wedging and slipping, develop simultaneously.

In its lower part, the soil behind the sheeting expands; its density decreases and surfaces of sliding develop in it. In the upper part, however, under normal conditions of deformations of the braces of the cut, a densification of the soil takes place and a wedging of its hard particles, i.e., the so-called "arching action" *(Ref. 2)*.

In spite of the absence of a mathematically rigorous justification, Terzaghi's design trapezoid *(Fig. 1)* fully justified itself in practice when cuts were braced by horizontal struts. However, experience with inclined struts proved to be somewhat different in certain cases.

When the width of a cut is very large, the bracing of its walls by horizontal struts *(Fig. 1)* is not practical. For that reason, one frequently uses the method shown *(Fig.*

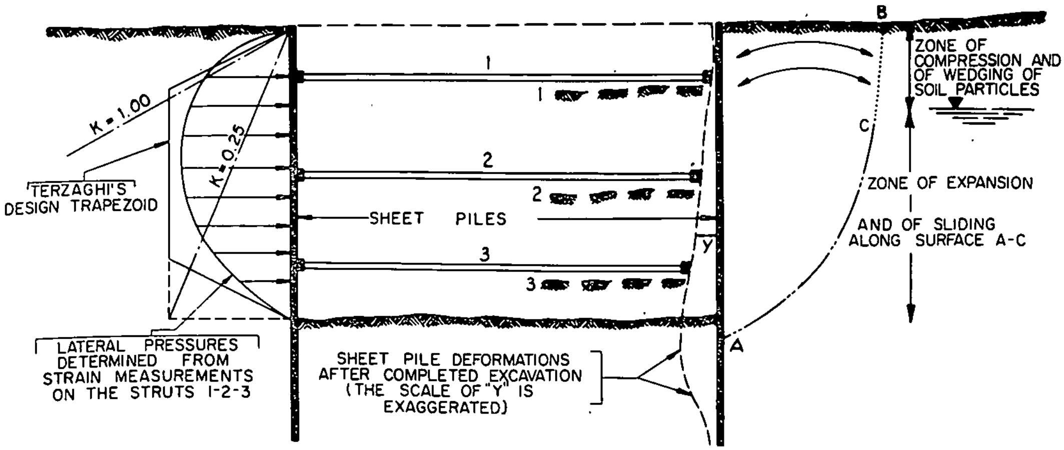

FIGURE 1

2-1), that is the excavation is performed within sheet pile walls approximately along the natural slope b-b, with a step-wise lowering of the ground water level by well points. After that, the central part of the foundation of the structure is concreted and the upper inclined strut N_1 is installed. The lower horizontal strut H_1 is installed after further lowering of the water level and excavation of the sloped soil.

The horizontal component L_1 of the pressure on the upper strut N_1 is determined in the usual way from the design diagram of lateral soil pressures accepted for the case handled. The vertical component V_1 is determined as a function of L, and of the inclination of N_1. Usually the value V_1 equals only part of all tangential active stresses T_a which are developed along the

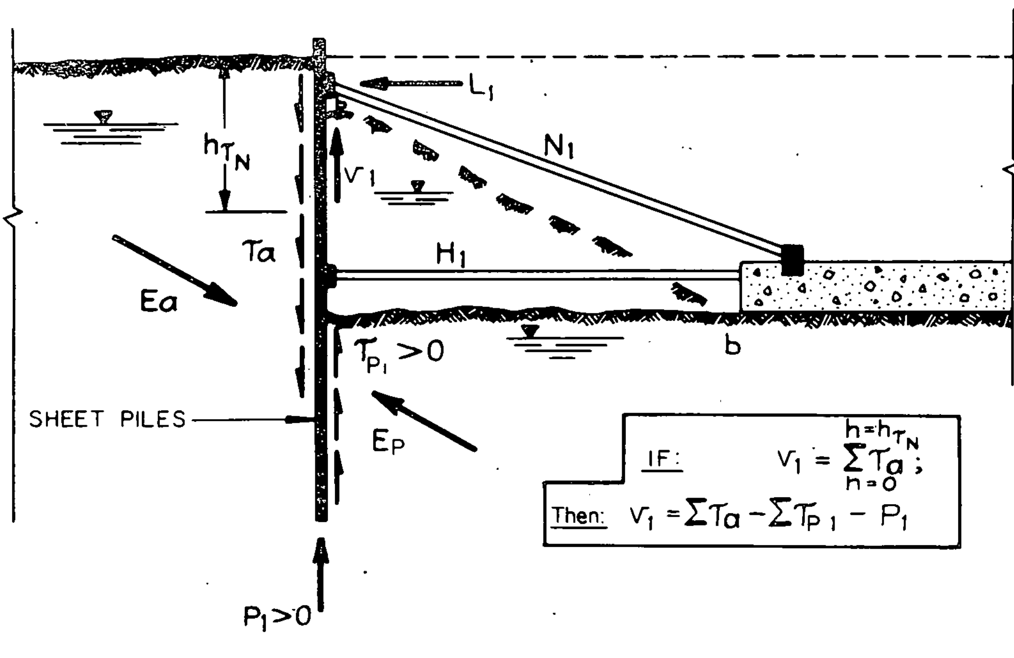

FIGURE 2-1

face of the sheet pile wall by the inclined active pressure of the soil E_a, i.e.:

$$V_1 = \sum_{h=0}^{h=hT_N} T_a \qquad (1)$$

The rest of the active tangential stresses T_a is balanced by passive tangential stresses T_{P1} *(Fig. 2-1)* and by the resistance P_1 of the tip of the sheet piling, i.e.:

$$V_1 = \Sigma T_a - \Sigma T_{P1} - P_1 \qquad (2)$$

If for any reason the tangential passive stresses T_{P2} and the resistance of the tip of the sheeting P_2 will be reduced to zero then, as shown *(Fig. 2-2)*, the value of the vertical component V_2 of the inclined strut N_2 will be greatly increased as compared to the usual value V_1, since:

$$B_2 = \Sigma T_a \qquad (3)$$

The horizontal component L_2 of the inclined strut will be increased correspondingly and equilibrium will become possible only as a result of the development in the upper part of the soil behind the sheeting of lateral pressures of a passive character which have to be greater than the values of the design trapezoid of Terzaghi shown in Figure 1. The strut N_2 itself will then be subjected to compression which will exceed considerably the design values, a circumstance which may have dangerous consequences.

With the help of the two following diagrams, let us now consider the question as to what kind of circumstances may produce in practice such an undesirable situation. If soft deposits extend below the level of the bottom of the excavation, then the foundation has to be supported by piles which, as well as the sheet pile wall, are then driven into a more compact underlying layer as this is diagrammatically shown *(Fig. 2-3)*.

The driving of piles for the outer parts of the foundation is carried out after completion of the excavation. If the saturated soil below the level of the bottom of the excavation is primarily of a non-cohesive type, e.g., it consists of silty fine-grained sands or of rock flour, then the vibrations which accompany the driving of piles in the immediate vicinity of the sheet pile wall produce a temporary but considerable decrease of the forces of inner friction in the soil.

As a consequence, this produces also a decrease of its tangential passive stresses T_{P3} along the surface of the sheeting and of the resistance P_3 of the tip of the sheeting. This decrease of T_{P3} and of P_3 has to be balanced by an appreciable increase of

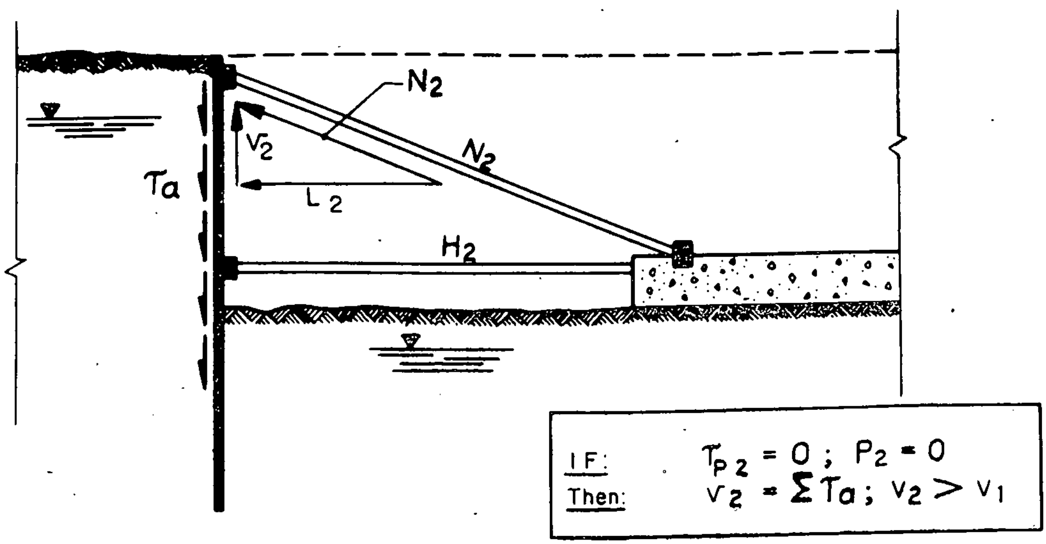

FIGURE 2-2

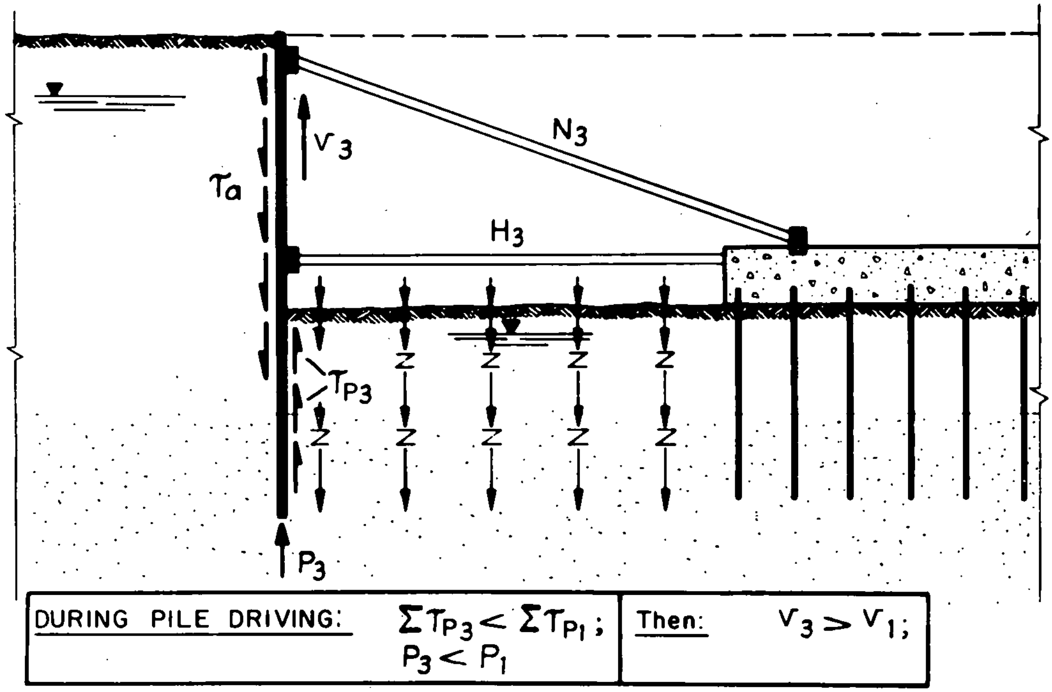

FIGURE 2-3

the vertical component V_3 and hence also of the axial pressure N_3 on the inclined strut.

Another circumstance with similar undesirable consequences is diagrammatically indicated *(Fig. 2-4)*. The difference between the active and passive lateral pressures (p_a—p_b) along the span "L" of the sheet pile wall will develop in it bending moments M. These moments may be increased by an eccentricity of the tangential active stresses T_a along the sheeting caused by the lateral displacement Δy of the upper part of the sheet pile wall towards the interior of the cut during its excavation and prior to the installation of the upper inclined strut N_1, see Figure 1.

Given sufficiently large values of the span "L," of the eccentricity "y" and of the pressures (p_a—p_b), the development of a moment M_o in the sheeting is possible, whereby this moment M_o may exceed the moment M_{PL}, after which a flow and plastic deformations of the material of the sheeting may take place. As a result the sheet piling is no longer in a position to transmit axial loads to the underlying compact soil. Therefore the inclined strut N_4 has to resist the greater part of the active tangential stresses T_a.

In my capacity of associate of the firm of King & Gavaris, consulting engineers, I participated in the study of a case which occurred in a deep cut (h = 17 m) in waterlogged glacial deposits of varved clays, silty sands and of rock flour where apparently took place simultaneously the conditions illustrated both by Figure 2-3 and by Figure 2-4, i.e: the driving of piles, the decrease of the passive resistance p_p and the corresponding increase of the bending moment M_o accompanied by a decrease of the tangential passive stresses T_p. The situation was aggravated by the fact that only the individual master piles and not the entire sheet pile row reached the underlying layer of compact sand and gravel.

What should be the preventive measures in such cases? First of all, the ground water lowering should be performed not only by stepped rows of well points inside of the cut, but also with the help of individual deep wells with pressure pumps at their bottoms placed outside of the sheet pile wall. Second, additional rows of struts should be

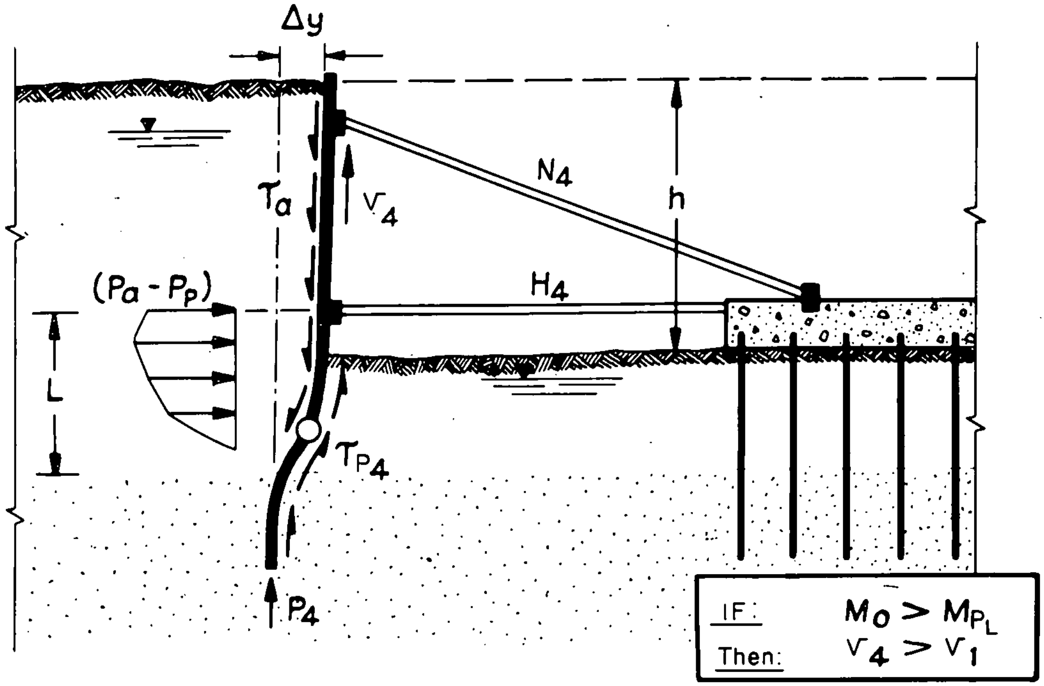

FIGURE 2-4

installed having the purpose to decrease the span "L" and to relieve the upper row of inclined struts. Third, consideration should be given in design even if only approximately to all these special factors when computing and detailing the structure.

A rigorous mathematical analysis presents here even greater difficulties than in the design of horizontal struts. For that reason additional measurements on construction sites of this type are greatly to be desired.

(2) Design of Anchored Sheet Pile Bulkheads

A number of experiments performed during the past 15 years have resolved certain contradictions between various design methods of anchored sheet pile bulkheads.

The oldest and best known of these methods did not consider any fixation of the lower part of the sheeting in the soil, see diagram of lateral pressures *(Fig. 3-B)*, and the corresponding curve (1) of the diagram of bending moments *(Fig. 3-A)*. In reality such diagrams are possible only in two cases: first in the case of absolute rigidity, i.e., non-flexibility of the sheeting, see the straight line (1) *(Fig. 3-C)*, and second, in the case of limit equilibrium of the entire soil into which the sheeting is driven, i.e., when the safety factor of the passive resistance of the soil has a value equal to unity along the entire depth of embedment of the sheeting. This latter case obviously is not permissible for the design of any kind of structure.

At the beginning of the 1930's, the German engineer Blum *(Ref. 3)* developed a method for the evaluation of the fixation of the lower part of the sheeting and proposed a simplified method of design with the help of the so-called "equivalent beam." The method consisted in the determination of the point at which the bending moments are equal to zero.

An imaginary "hinge" was assumed at that point and the part of the sheeting above it was designed as a statically determinate beam on two supports. The point of zero moment, i.e., the hinge, was again determined on the basis of a state of limit equilibrium of the soil along the entire depth of embedment of the sheeting. As a result,

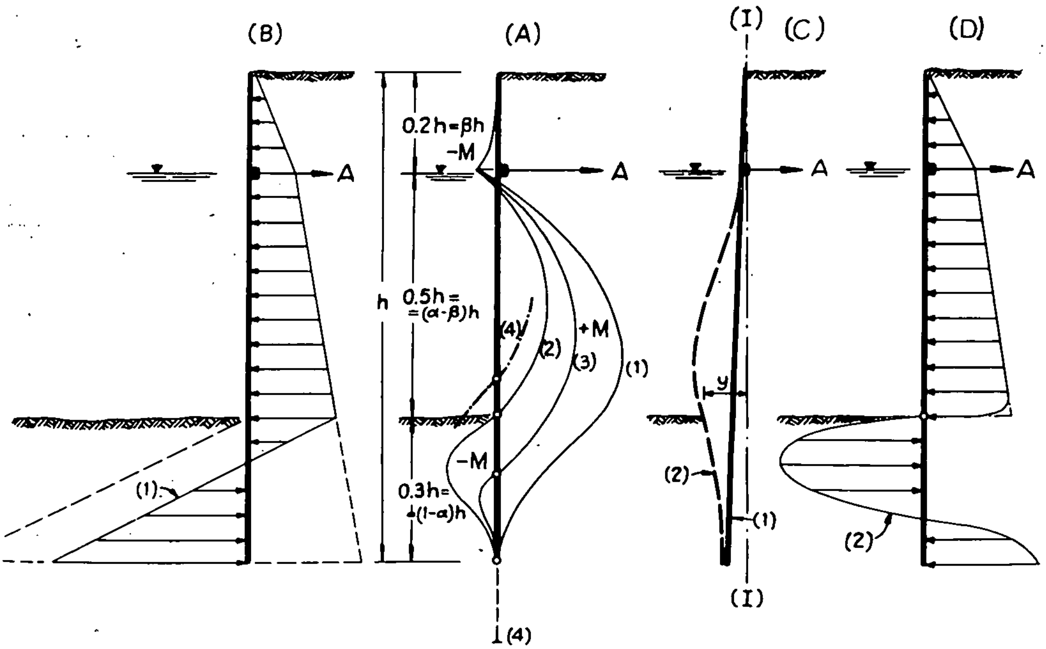

FIGURE 3

only the value of the angle of internal friction of the soil influenced the location of the hinge and the required depth of embedment reached appreciable values. The degree of flexibility of the sheeting was *not* taken into consideration.

In 1943-49, I had the opportunity to direct model tests at Princeton University for the Bureau of Yards and Docks of the U. S. Navy. These tests showed *(Ref. 4)* that:

1. The redistribution of active pressures as suggested by the Danish rules does not actually occur.

2. Consideration of passive pressures above the anchor level according to the 1938 suggestion of the German scientist Ohde corresponds to reality only in the case of absolute rigidity of the anchor and of its supports and only in the case of dredging of the soil in front of the sheet pile bulkhead and not of backfilling behind the bulkhead.

3. The fixation of the lower part of the sheeting embedded in sand takes place under a much smaller depth of penetration than followed according to Blum, see curve (2) *(Fig. 3-A)*.

4. At the same time the "hinge" is located in the immediate vicinity of the dredge level of a sandy harbor bottom in front of the sheeting if the latter has a flexibility which corresponds to steel sheet piles having a cross-section in the form of the letter "Z" as used in the USA.

The latter two circumstances are explained by much higher passive resistance of the sandy soil *(Fig. 3-D)* than was considered possible previously. This is brought about by the deflection "y" of the sheeting *(Fig. 3-C)*. Further tests of my associates at Princeton in 1951-53 *(Ref. 6)* have confirmed this explanation.

Measurements in the field also showed *(Ref. 8)* that an increase of the depth of embedment raises the level of the "hinge," see the curve (4) *(Fig. 3-A)*.

In 1952-57 the talented young English scientist, Peter Rowe, published *(Refs. 5, 7)* data concerning his tests which confirmed the results obtained by us at Princeton and which developed them appreciably further.

Having decreased the scale of his model, approximately to 1:25, whereas the model at Princeton had the scale of 1:10, and using dry sand instead of the submerged sand with which we worked, Peter Rowe was able to increase more than tenfold the number of tests he carried out as compared to our own. Further, he established definite relationships between the flexibility of the sheeting, the density of the sand into which the sheeting is embedded and the maximal bending moment (+ M) of the sheeting *(Fig. 4)*. To that end Rowe introduced the coefficient of flexibility ρ:

$$\rho = \frac{H^4}{EI} \tag{4}$$

where:

- H = length of the sheeting in feet *(see Fig. 3)*.
- E = modulus of elasticity of the material of the sheeting in psi.
- I = moment of inertia of the sheeting in inches^4 per foot width of the sheeting.

The computation of this coefficient in the metric system should not present difficulties.* The moments measured were expressed by Rowe in per cent of the value of the moment M_1 without any kind of fixation of the lower end of the sheet pile, see curve (1) *(Fig. 3-A)*. Typical results are shown on Figure 4 for coefficients $\alpha = 0.7$ and $\beta = 0.2$, i.e., for the same depth of embedment and location of the anchor as shown in Figure 3. At the same time the surface surcharge $p_s = 0$.

* 1 foot = 0.305 m.; 1 inch = 2.54 cm..; 1 lb. = 0.454 kg.

Peter Rowe suggested plotting on the same diagram the curves of resistance moments of various types of sheet pile sections for a given permissible value of their bending stress. The intersection of such a curve of resistance with a curve of experimental moments for the given type of soil indicates the most rational use of permissible stresses for a given sheet pile section. A design is carried out in practice by trial selections of sections and their verification on a diagram of the type shown on Figure 4.

The curve (A) on Figure 4 shows the curves of resistance moments to bending of steel sheet piles of the USA sections MZ-27, MZ-32 and MZ-38 for a permissible stress of 22,000 psi = 1560 kg/cm^2.

On the same Figure 4 is shown the value of the moment M_2 (= 45% of M_1) obtained in accordance with the simplified method of the "equivalent beam" with a hinge at the level of the dredged bottom of the harbor, see curve (2) *(Fig. 3)*. This line intersects almost at the same point the resistance curves (A) and the average experimental curve of Rowe. Thus the results of all these tests fully agree with each other.

The method of Rowe greatly facilitates understanding of the fundamentals of the problem. Thus if we plot on Figure 4 the resistance curve (B), for the same sheet pile sections for which we have plotted the curve (A), but for the additional deflection which would correspond to the first limit of elasticity of the metal of 33,000 psi = 2360

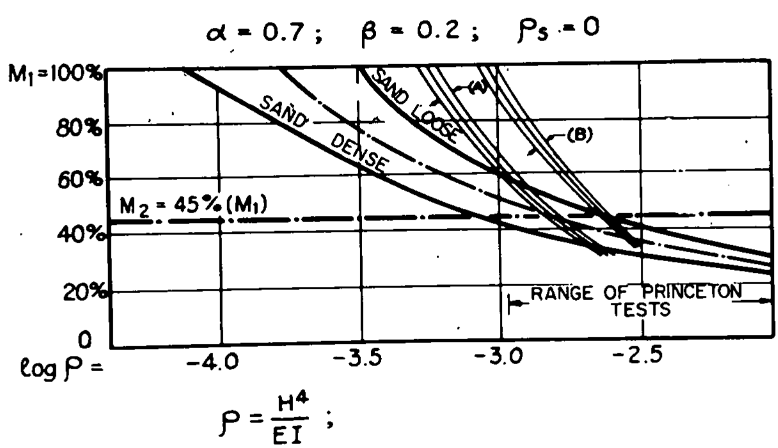

FIGURE 4

kg/cm², then the corresponding experimental moment will decrease from 45 per cent of M_1 *(Fig. 4-A)* to 36 per cent of M_1 *(Fig. 4-B)*. In other words, this indicates the presence of an additional factor of safety equal to $45/36 = 1.25$ if one designs a steel sheet pile wall for the stresses of 1560 kg/cm² usually considered permissible in the United States.

Rowe worked out experimentally a system of curves similar to those of Figure 4 for clays as well *(Ref. 7)*. However, more complicated soil conditions have not yet been taken care of by this semi-empirical method.

Measurements which I carried out in a number of harbors have shown that soil conditions corresponding to simplified laboratory conditions of model tests should be considered an exception rather than the rule. There are reasons to assume that the assumption for the design of average values of coefficients obtained from tests with two different soils will not always be a correct solution. Thus, for instance, as shown *(Fig. 5)*, a sheet pile wall driven through a layer of plastic clay into compact sand will be subjected to an appreciably larger curvature and therefore also to an appreciably larger bending moment so that its "hinge" will be located lower than in the case of a wall which does not reach sand. At the same time, however, the lateral displacement of the latter at the dredge level will be larger than the displacement of the first, i.e., $y_c > y_s$.

Attempts of mathematical analysis of the problem combined with the use of coefficients of lateral subgrade reactions of different soils, determined on the basis of laboratory tests, are being made but have not yet led to practicably usable solutions.

In any case, any theoretical solution will need verification by full-scale field measurements. The location of the point of zero moment, i.e., of the imaginary "hinge," is easily determined *(Refs. 8, 9)* and therefore represents a reliable check criterion of the

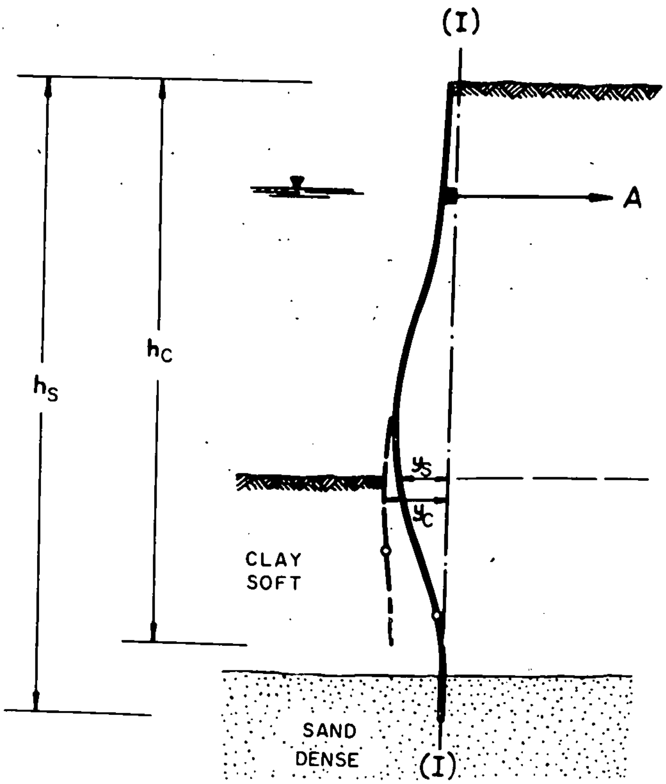

FIGURE 5

degree to which any theory corresponds to real conditions. Therefore, I consider that the simplified method of the "equivalent beam" with modifications reported by me at last year's conference in Brussels *(Ref. 9)* represents a practical, feasible approach to a final solution of the design problems for this type of structures.

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Critical Elements of Design and Construction of Heavy-Duty Flexible Pavements

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The increase in weight of aircraft has made it necessary to devote special attention to the design and construction of flexible pavements that will perform satisfactorily under these unprecedented aircraft loads and operational characteristics.

The load on a single wheel of an aircraft can now be as high as about 60,000 to 70,000 lbs. The number and alignment of the wheels vary with the gross weight and type of aircraft. In order for the wheels to carry such loads without using excessively large tires, the inflation pressures of the tires must be much higher than those required in the past. Tire pressures are now approaching 300 psi and this intensity of contact loading makes it necessary to give special attention to the upper portion of the pavement.

In addition to the great increase in load weights and tire inflation pressures, the operational characteristics of the aircraft while on the ground must be considered. Almost all recently developed aircraft can be steered by mechanical means, which results in operation of the newer aircraft in a path much narrower than that of the older aircraft which are steered by rudder alone. Such concentrated traffic, referred to as channelized traffic, has the effect of deteriorating the pavement in a shorter period of time than traffic distributed over a wide area. While certain areas receive the concentration of channelized traffic, other areas are subjected to lesser or even quite light traffic, or only to traffic of aircraft not loaded to maximum weight.

Basic Design Requirements

A flexible pavement is constructed in layers of material placed on a subgrade. Each layer must have adequate strength to resist the stresses imposed on it. As the intensity of stress is highest at the surface and decreases with depth, it follows that each layer must have a higher strength value than the layer immediately below it. Stated in another way, a layer of a given strength must have a certain thickness of a higher-strength material above it to provide adequate protection from imposed loads.

In addition to adequate shear strength in the entire pavement structure, a high degree of compaction in all layers must be obtained to insure that additional densification does not occur under the traffic of heavy aircraft. The high degree of compaction not only is essential to insure the maximum strength of the various layers but also is quite essential to insure satisfactory surface smoothness during the life of the pavement. Differential compaction can produce an undesirable surface condition which will have detrimental effects on the operation of heavy aircraft. Aircraft with bicycle-type landing gear are especially sensitive to roughness and unevenness of runway surfaces.

A high compaction effort is required in the field on base courses in order to meet the design requirements. After this initial density has been obtained, the surface of the layer of base course in question is subjected to about 30 coverages of a very heavy rubber-tired roller having a tire pressure ranging from 90 to 150 psi to insure that a uniform and high degree of compaction has been obtained and that no isolated uncompacted areas exist. This latter procedure is referred to as "proof-rolling."

Generally speaking, the densification of the base course shall be at least 100 per cent laboratory modified AASHO (American Association of State Highway Officials) density. Quite often it is found that with the addition of proof-rolling, the density will be as high as 104 per cent of modified AASHO density.

A considerable amount of information has been obtained on heavy-load prototype traffic in the field, and this information indicates that it is not unusual for a base course to exist at 105 per cent modified AASHO density as a result of compaction by traffic. The degree of compaction of the subbase course usually should be as high as that of the base course, and proof-rolling on at least the surface of the top layer of the subbase course is required.

Below the subbase course in the subgrade, the compaction requirement decreases until at a depth of about 5 to 6 ft below the pavement surface a compaction of 80 to 90 per cent modified AASHO density is required.

The surface smoothness requirements are quite rigid in that surface deformities must be such that they will not produce more than ⅛-in. variation under a straightedge 12 ft in length. Proper initial densification is essential to insure not only that this high degree of surface smoothness exists at the time the pavement is constructed but also that it is maintained during the life of the pavement as the base and subbase materials are subjected to possible further densification by traffic.

The effect of the blast (and heat) and the spillage of fuel from jet aircraft must be considered in the design of pavement surfaces; this, of course, is in addition to the ordinary requirements that the bituminous concrete surface wearing course must be resistant to wear and weather. In areas subjected to repeated blasts, such as maintenance areas and the ends of runways, portland cement concrete is generally used. Parking areas which are subjected to some fuel spillage and occasional blast are constructed of either portland cement concrete or rubberized-tar flexible pavements. The rubberized-tar pavement has somewhat higher resistance to blast and is much more resistant to fuel spillage than asphaltic concrete.

Other Design Considerations

It has been found desirable to place layers of a base course or a subbase course by means of mechanical spreaders to insure a layer of uniform thickness. This uniformly thick layer tends to produce more uniform compaction and also gives much greater insurance of meeting grade limitations and thus producing a pavement surface which initially meets and will maintain smoothness requirements.

The quality of the aggregates used in bases and subbases must be such that the aggregates will meet ordinary soundness and durability requirements. It has been found that the denser-graded material with a maximum size not greater than 2 in. is the most desirable. Probably the most critical quality requirement of a base or subbase material is the plasticity index. It is essential that the base course layer be nonplastic, that is, have a plasticity index not over 5. The plasticity index of subbase materials may be slightly higher, but under no conditions should it be over 12.

The design requirements of the materials below the upper subbase are not discussed here because it is evident that the basic design principles allow the use of lower-quality materials at the greater depths. The quality and strength of the materials at any given depth in the pavement determine the total thickness of material above the given layer. Similarly, the degree of compaction of the lower subbase course and subgrade materials decreases with depth.

Design Methods

The U. S. Army Corps of Engineers utilizes the California Bearing Ratio (CBR) system *(Fig. 1)* in the over-all design of a flexible pavement structure. The strength of each type of material is determined by means of the CBR test. By means of test sections and by observations of pavements of airfields, the Corps of Engineers has developed a relationship between the CBR of a material and the thickness of pavement required above it to protect the layer from the imposed loads.

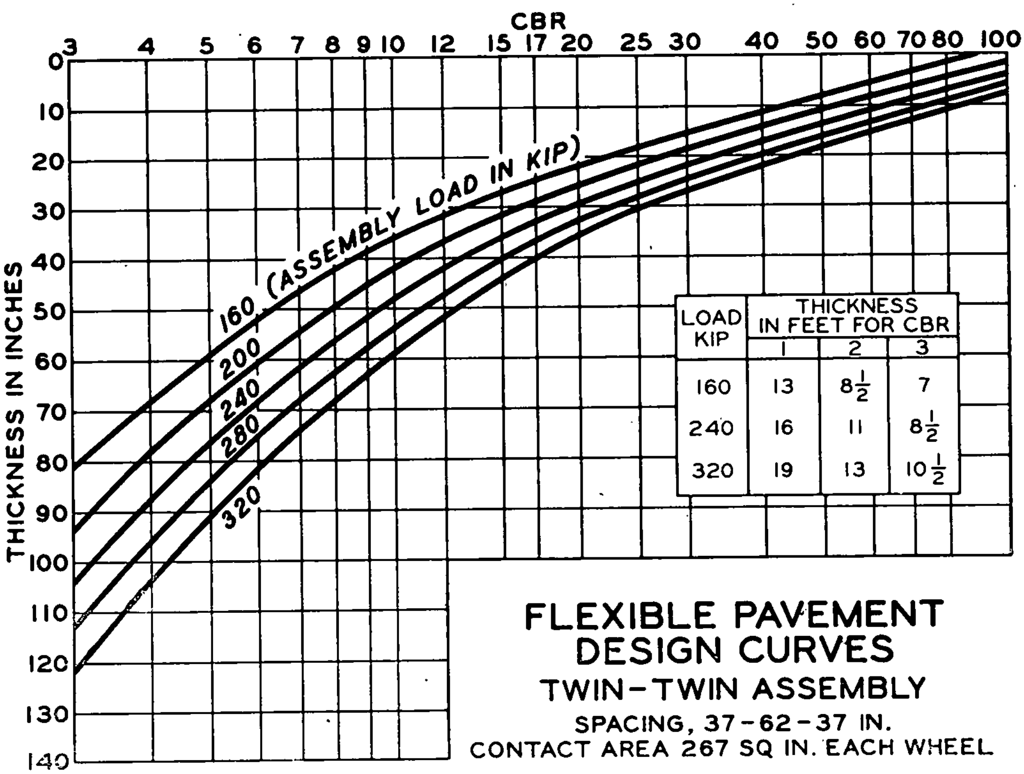
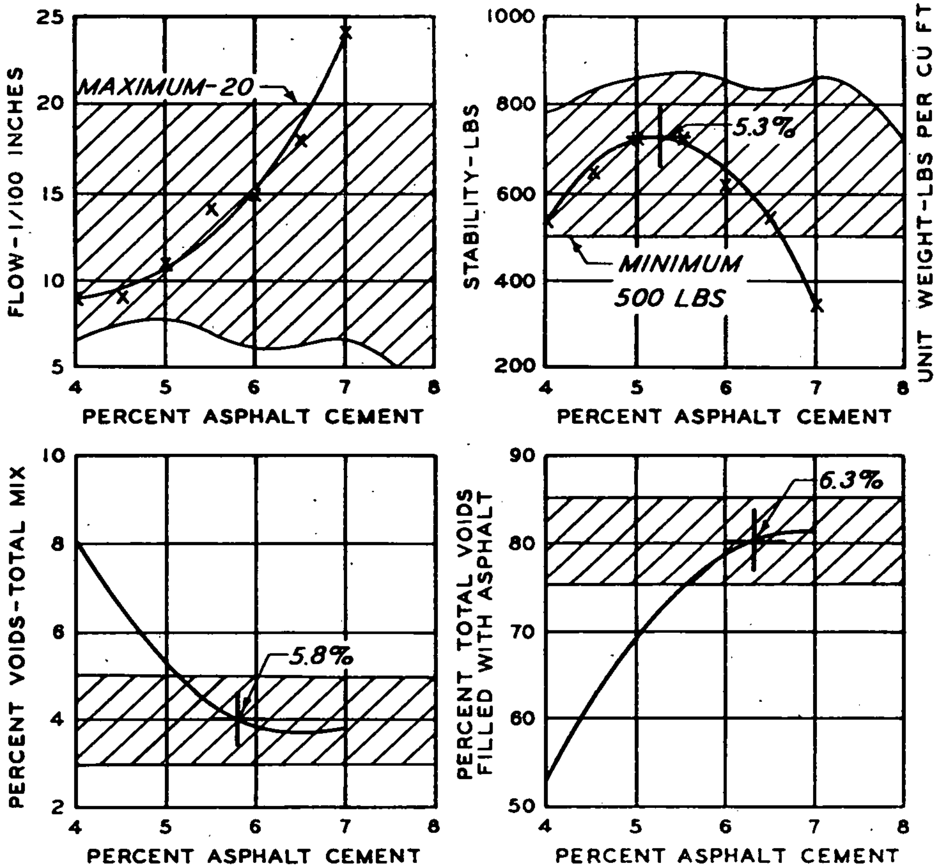


FIGURE 1



TEST PROPERTY	SELECTED ASPHALT CONTENT-PERCENT
STABILITY	5.3
UNIT WT-TOTAL MIX	5.5
VOIDS-TOTAL MIX	5.8
VOIDS FILLED WITH ASPHALT	6.3
AVERAGE	5.7

TYPICAL CURVES TEST PROPERTIES VS ASPHALT CONTENT

ASPHALTIC CONCRETE

FIGURE 2

The Marshall procedure *(Fig. 2)* is used for designing bituminous concrete surfaces of flexible pavements. The bitumen content for light-duty pavements and for the portions of heavy-duty pavements that receive relatively light traffic is determined by the conventional Marshall test. This bitumen content is decreased for the areas of pavement subjected to channelized traffic of heavy aircraft. This latter is necessary because of the high compaction effects of the intensive traffic of heavy airplanes and the nonfeasibility of increasing the laboratory compaction effort to duplicate this high densification produced by prototype traffic.

The objectives of the Marshall method of design of bituminous surfaces are to insure that the mixture has sufficient voids to prevent plastic movement (flushing) when the pavement is consolidated under prototype load and that the bitumen content is high enough to insure good durability and weathering characteristics of the pavement.

It has been found by experience that these criteria can best be met when a sound and well-graded aggregate is used. Assuming the use of high-quality materials, it is considered that the proper proportioning of the asphaltic cement is the most critical feature in the design of satisfactory bituminous mixtures. Basically, the Marshall method is developed so as to furnish the maximum asphaltic content in a mix which will not flush under traffic.

Heavy-load bituminous pavements are constructed with conventional machinery and equipment. Recently it has been found that the use of medium-heavy pneumatic-tired rollers, in addition to steel-wheel rollers, is of considerable benefit.

Equipment and Procedures

In the last few years, real "breakthroughs" have occurred in compaction equipment development and/or procedures, the principal ones being as follows:

(a) Heavy rubber-tired rollers (90 to 150 psi) weighing 32 tons and above produce a high degree of compaction in bases and subbases. The 32-ton self-propelled equipment is particularly efficient in operation.

(b) Medium-weight rubber-tired rollers (90 to 150 psi) weighing 16 to 32 tons are very efficient for compaction of asphaltic concrete. High densities are easily obtained. The self-propelled feature is particularly effective and efficient.

(c) The use of asphaltic mixture spreaders for upper subbase and base course layers has proved particularly effective in furnishing true grade, thus insuring a smooth surface and improving uniformity of compaction.

(d) Proof-rolling with heavy rubber-tired rollers when properly specified and executed is very useful in producing satisfactory heavy-duty flexible pavements.

The large size of all the pavement facilities at a heavy-duty air base means that large quantities of earth must be moved during the course of construction. To move such large quantities of earth, high-performance earth-moving equipment must be used. The equipment must carry a large pay load, travel rapidly, and be capable of close control both in loading and unloading operations.

Summary

It is believed that if proper cognizance is given to the basic design considerations discussed, as well as to the other design considerations noted in this paper, a satisfactory heavy-duty flexible pavement can be designed by utilizing the CBR method for the over-all pavement and the Marshall method for the bituminous concrete. However, good equipment and construction procedures are necessary to insure that specification requirements are met and satisfactory pavements constructed.

Insofar as load-carrying capacity is concerned, it is considered that flexible pavements are entirely satisfactory for the interior portions of runways of heavy-duty airfields. Further, it is believed that, exclusive of blast and fuel-spillage areas, satisfactory flexible pavements can be designed for the heavier-traffic areas of runways and taxiways. However, some additional field and laboratory research is needed in this area.

Summary of Rotary Cone Penetrometer Investigations

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The natural meandering tendency of the Lower Mississippi River causes major bank failures in certain types of river deposits. These failures have threatened the levees at certain locations and increased the problem of stabilizing the river banks for navigation and development purposes. In 1947, extensive potamology investigations were started to study revetment failures, their causes and methods of preventing them, and to develop new methods of riverbank stabilization.

During the period 1947 to 1952, soils investigations established that all the major bank failures occur in point bar deposits composed of a relatively thin stratum of more or less cohesive overburden materials underlain by a relatively thick stratum of fine sand. The failures, while of short duration, are progressive rather than entirely instantaneous. They are composed of a series of blocks which individually fail instantaneously, and are believed to be associated with partial to complete liquefaction of the fine sand.

It is believed that excess pore pressures develop in strata of the fine sand at subcritical densities, probably as a result of strain in the bank produced by a shear failure of the underwater toe caused by scour due to current attack, and this results in liquefaction or flow-type failures. The principal characteristics of the area after failure are:

1. Comparatively large in area.
2. Scallop-shaped with a narrow to quite narrow opening on the river side through which material flows.
3. Trees on the bank fall with the tops riverward, which is the opposite of that in ordinary shear slides.
4. Bank slopes after failure are much flatter than that which could be predicted to be unstable by conventional analysis.

As a part of the 1947-1952 program, a rotary cone penetrometer was designed to investigate economically the relative density of deep sand deposits along the river banks. Preliminary field tests with the cone penetrometer had shown promise and, therefore, beginning in 1957 additional cone studies were initiated to determine the effect of surcharge pressure, gradation and shear strength of sands on the cone thrust, with the primary objective being to develop a reliable relationship between cone thrust and relative density of the in situ fine sand deposits.

Program of Tests

The test program conducted to date has consisted of three series of tests. In the first series, cone penetration tests were performed on a saturated fine sand confined in a steel tank at various relative densities and subjected to various surcharge pressures to determine the effect of surcharge pressure and relative density on cone thrust.

In the second series of tests, undisturbed samples were obtained from a fine and a medium sand contained in a steel tank at various relative densities and subjected to various surcharge pressures to determine change in density caused by sampling. In both the first two series of tests, pressure cells were used to determine normal pressure in the sand specimens at depth in the tank.

In the third series of tests, cone penetration tests were made in a deep deposit of fine sand at a typical site along the Mississippi River. Undisturbed samples of the sand were obtained adjacent to each cone penetration. The purpose of the third series of tests was to compare the relative density predicted by the cone penetrometer with the relative density measured from undisturbed samples.

A fourth series of cone penetration tank tests is now under way to determine the

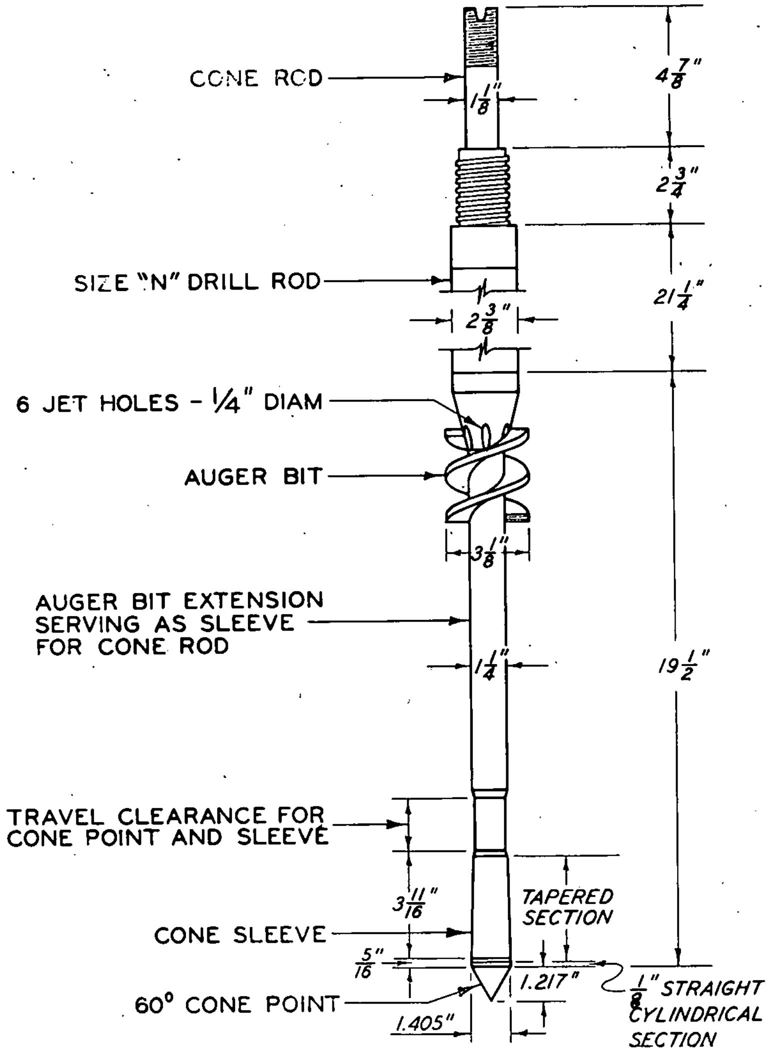

FIGURE 1
Rotary cone penetrometer.

effect of sand gradation and shear strength on cone thrust.

Cone Penetrometer

A sketch of the rotary cone penetrometer used in these investigations is shown *(Fig. 1)*. It is designed for use with a truck-mounted rotary drilling rig and consists of a sounding rod having a conical point with a base area of 10 cm^2 and a sleeve which fits over a downward extension of the hollow stem of a helical auger bit on the drill rod. The cone rod is tubular above the auger bit and extends up through the drill rod and a swivel to a proving frame attached by tie rods to the flange of the swivel.

The proving frame has a maximum capacity of about 9,100 lbs. During the operation, drilling fluid flows down the inside of the cone rods and emerges from vents above the auger. The auger is rotated during the advance but the cone does not rotate. The force on the cone is transmitted by the cone rods to the proving frame. The distance between the bottom of the auger and the cone tip was 15½ in. for these investigations.

Cone Penetrometer Tank Tests

The purposes of the first series of tests were to find the relationship between cone thrust and overburden pressure for various densities of one type of sand and to determine the minimum thickness of loose or dense strata which can be reliably detected by the cone penetrometer.

The investigation consisted of a number of calibration tests and cone penetration tests performed on specimens of saturated fine sand confined in a steel tank 6½ ft high by 3½ ft in diameter. Surcharge loads up to 100 psi were applied to the top of the specimens by three hydraulic jacks. A view of the cone penetrometer and assembled test apparatus is shown *(Fig. 2)*.

Calibration tests were made to determine the decrease in normal pressures with depth developed within the tank filled with fine sand placed at relative densities of 90, 65, and 40 per cent. Penetration tests were made on specimens constructed to the same densities as the calibration test specimens and on loose specimens with 12-, 6-, and 3-in. dense built-in strata, and on dense specimens with 12-, 6-, and 3-in. loose built-in strata. All specimens were subjected to 30-, 60-, and 100-psi surcharge pressures during testing.

The relationships between cone thrust and normal pressure for the densities of sand tested are shown *(Fig. 3)*. Since the overburden pressure in the field can be estimated, it should be possible to determine the density in the sand tested by measuring the cone thrust and computing the overburden pressure on the basis of the depth and average unit weight of the material. The tests also showed that the presence of 12-, 6-, and 3-in.-thick built-in strata could be detected, although a 6-in. stratum probably is the minimum thickness for which the relative density may be reliably obtained. Thus the correlations desired were determined for a fine sand.

Change in Density Caused by Sampling

The second series of tests consisted of undisturbed sampling of a fine sand and a medium-fine sand, contained in the same tank used for the penetration tests. Four

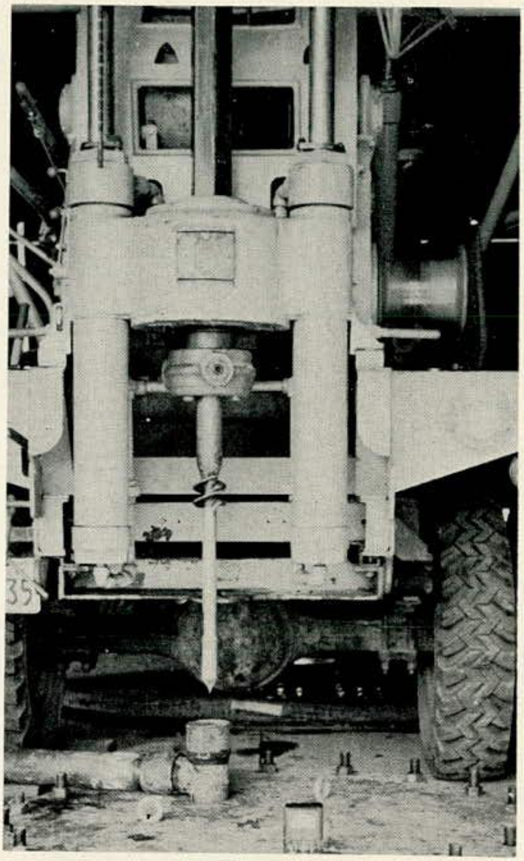

FIGURE 2
View of cone penetrometer and assembled test apparatus.

specimens were prepared, two of fine sand placed at relative densities of 20 and 90 per cent, and two of medium-fine sand placed at relative densities of 24 and 90 per cent.

The relative densities were chosen to simulate the extremes of natural densities suspected to exist in the field. Pressure cells were placed in the specimens to determine pressures at certain depths within the sand placed at various densities and to determine the changes in pressure during sampling. The specimens were subjected to surcharge pressures up to 100 psi.

Three 30-in.-long undisturbed samples, one at each of three surcharge pressures, were obtained from each specimen with a 3-in.-diameter Hvorslev fixed-piston thin-walled, steel tube sampler. The samples were cut into 3-in. increments and the variation of change in density was determined with respect to surcharge pressure and

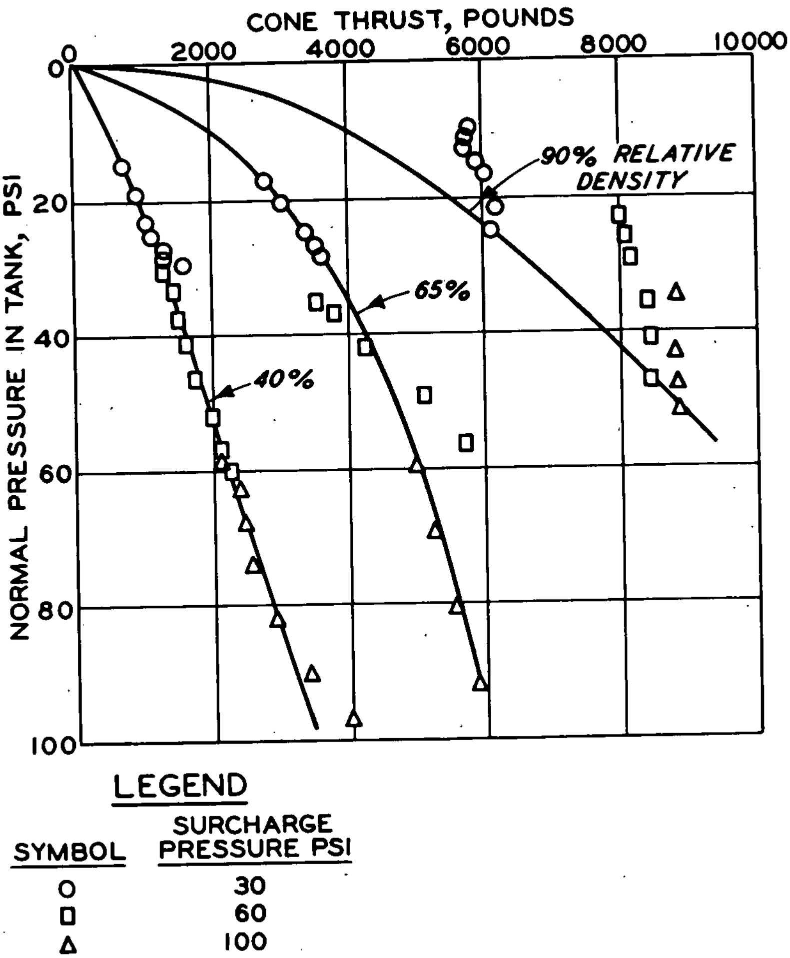

FIGURE 3
Cone thrust vs normal pressure in tank for fine sand at various densities.

height of increment in the sample tube.

The analysis of data indicated that the change in density during undisturbed sampling of sand is dependent on overburden pressure and location within the sample tube. In general, loose material increased in density and dense material decreased in density during sampling.

Correction factors, based on a comparison of the measured density of sampled increments with placement density, were developed for both overburden pressure and location of the increment in the sample tube. The combined correction factor generally amounted to less than 1 pcf, or about 5 to 6 per cent in relative density for the middle 18 in. of the 30-in. sample for conditions commonly encountered in the field. It was concluded that the change in density which occurs during undisturbed sampling of sand is not a serious problem.

Field Investigation

A field investigation was conducted on a Mississippi River bank to determine if cone penetration data obtained from the tank tests could be used successfully to determine the relative densities of deep sand deposits in the field. Two undisturbed sample borings and two cone penetrations were made in an area where the thickness of fine sand was known to be at least 35 ft. One undisturbed sample boring was located 5 ft from each cone penetration, and 30-in.-long undisturbed samples were obtained at 3½-ft intervals of depth, using the same sampler

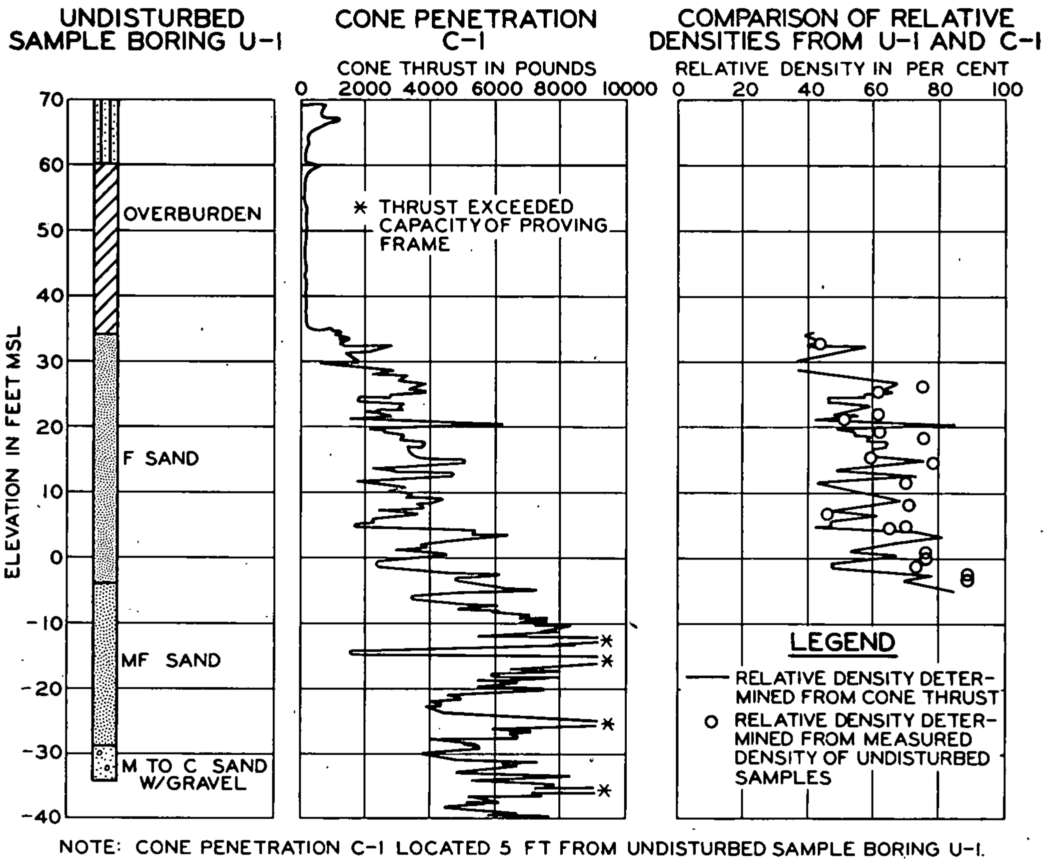

FIGURE 4
Data from boring U-1 and cone penetration C-1.

as that described previously. Cone penetration resistance was measured for each 0.1 ft of penetration.

Data from one undisturbed sample boring and one cone penetration are shown *(Fig. 4)*. The undisturbed field samples were cut into 3-in. increments and the densities determined. The relative densities of selected 3-in. increments were determined by laboratory test and these data are shown on the right-hand plot of Figure 4. The relative densities based on cone penetration resistance and laboratory correlations shown on Figure 3 are also shown on the right-hand plot of Figure 4.

The data indicate that the relative density of fine sand estimated on the basis of cone penetration resistance can be verified within about 10 per cent by the computed relative density determined from an undisturbed sample. The cone is sensitive to changes of relative density and shear resistance, and therefore should be considered an effective tool for estimating the shear resistance of deep sand deposits.

ABSTRACTS OF OTHER AMERICAN PAPERS

Brief resumés of subjects presented at the Soviet seminars by the exchange visitors follow. These are papers which had previously been presented or published in the United States.

Studies of Frost Problems in a Northern State

MILES S. KERSTEN, *Professor of Civil Engineering, University of Minnesota*

The paper reviewed the history in one state, namely Minnesota, of frost studies on highways, design of highways for frost conditions, and research studies which have been made at a university. The intent of the presentation was thus to give examples of attacks which are being made on frost problems without trying to cover the very broad subject of frost studies in the entire country. The information and most of the slides used were drawn from papers previously published.

The paper reviewed information on the winter climate in the United States and the manner of making soil surveys and designs for frost treatments in Minnesota *(Ref. 1)*. The findings of the Highway Research Board Committee on Load-Carrying Capacity of Roads as Affected by Frost Action concerning the loss of strength during the frost melt period were described, and the design thickness of flexible pavements in Minnesota was shown to indicate requirements in a frost area *(Ref. 2)*.

The laboratory studies for determination of thermal properties of soils made in Minnesota were described in some detail *(Ref. 3)*. Methods for calculating frost penetration in highways *(Ref. 4)* and field measurements for comparison with calculations made in Minnesota, were presented. *(Refs. 5, 6)*.

It was the intent of the paper to present short description of several phases of frost problems, and thus to suggest a variety of questions in the discussion. This was accomplished at the two seminars at which the paper and slides were presented.

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1. Braun, R. P. "Correlations of Soil Test Data to Design Relating to Highways and Airfields." Conference on Soil Mechanics and Foundations. University of Minnesota. 1959.
2. Meskal, G. A. "Final Report on Load-Carrying Capacity of Roads as Affected by Frost Action." HRB Bulletin No. 207. 1959 (also previous reports by same committee).
3. Minnesota Department of Highways. "Approved Flexible Pavement Design Standards." 1954.
4. University of Minnesota, Bulletin 128. "Thermal Properties of Soils."
5. Kersten, M. S. "Frost Penetration: Relationship to Air Temperature and Other Factors." HRB Bulletin No. 225. 1959.
6. Kersten, M. S. and Johnson, R. W. "Frost Penetration Under Bituminous Pavements." HRB Bulletin No. 111. 1955.

Soil Stabilization

T. WILLIAM LAMBE, *Professor of Soil Engineering, Massachusetts Institute of Technology*

At the seminars at Moscow and at Kiev, the writer gave a short description of some of the recent research conducted at the Massachusetts Institute of Technology on additives to improve the properties of soil. Presented were the results of work on additives to modify the frost susceptibility of soils and on additives to increase the effectiveness of portland cement as a primary stabilizer.

On the basis of laboratory tests the most promising additives for the modification of the frost heaving characteristics of soils were ferric chloride (an aggregant) and the polyphosphates (dispersants). A group of 11 dirty gravels were treated with 0.3 per cent of tetrasodium pyrophosphate and subjected to controlled laboratory freezing tests. The results of the tests showed that the phosphate reduced the rate of frost heave for all 11 gravels. The minimum reduction was to half of the untreated value, the maximum to essentially zero, and the average reduction was to 0.2 of the untreated value.

The material on frost additives presented at Moscow and Kiev came from a paper, "Modification of Frost-Heaving of Soils with Additives," by the writer published in Bulletin No. 135 of the Highway Research Board.

Research on the use of chemical additives to reduce the effectiveness of portland cement as a soil stabilizer was described. In general, the tests showed that, with virtually all soils studied, cement stabilization can be substantially improved (two to 10 fold) by the incorporation of relatively small quantities of sodium compounds which form insoluble compounds with calcium. The most beneficial additives were caustic soda, soda ash, sodium sulphite, sodium sulphate, sodium metasilicate and sodium aluminate. Optimum additive concentration was found to correspond very nearly to a sodium ion concentration in the molding water of 1.0 normal, i.e., between 0.5 and 2.5 per cent of the soil dry weight, depending upon soil and additive.

The soil cement work described in the Soviet Union came from two papers by the author (and associates) presented before the Highway Research Board Annual Meetings of 1957 and 1959.

SOIL STRUCTURE

In answer to requests by the Soviet soil engineers, the writer gave extemporary talks on soil structure at Kiev and at Leningrad. The nature of electrical and externally applied forces transmitted between adjacent soil particles was discussed. The variation of these forces with environmental conditions was next considered. Shear strength as dependent upon environmental factors and applied pressures was finally discussed.

Much of the material came from the writer's two papers on soil structure published in the May 1958 Journal of the Soil Mechanics and Foundation Engineering Division of the American Society of Civil Engineers.

Analysis and Design of Concrete Slabs on Ground

G. A. Leonards, *Professor of Soil Mechanics, Purdue University*

Conventional methods of analysis (utilizing Winkler's assumption, or an elastic half-space) were reviewed briefly. Reference was made to observations that for lightly loaded slabs (homes), or where the loads are temporary (pavements, industrial slabs on ground), warping caused by moisture and temperature gradients was sufficient to leave a portion of the slab entirely unsupported.

A new theory *(Ref. 1)* was outlined for calculating stresses and deflections in partly supported slabs of finite size. The significance of this theory in the analysis of concrete pavements was treated *(Ref. 2)*. Finally, the results of full-scale measurements *(Ref. 3)* for both upward and downward warping that confirms the validity of the new theory were presented.

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1. Leonards, G. A. and Harr, M. E. (1959). "Analysis of Concrete Slabs on Ground." Journal, Soil Mechanics and Foundations Division, ASCE, Vol. 85, No. SM3, p. 35.
2. Harr, M. E. and Leonards, G. A. (1959). "Warping Stresses and Deflections in Concrete Pavements." Highway Research Board Proceedings, Vol. 38, p. 286.
3. Wiseman, J. F., Harr, M. E. and Leonards, G. A. (1960). "Warping Stresses and Deflections in Concrete Pavements—Part II." Highway Research Board Proceedings, Vol. 39.

Recent USA Research on Soil Strength and Deformation Characteristics Under Dynamic Loading Conditions

H. BOLTON SEED, *Associate Professor of Civil Engineering, University of California, Berkeley*

This paper presented a review of research on the strength and deformation characteristics of soils under transient, repetitive and earthquake type loading conducted in the United States during the past 12 years.

The increase in strength of clays and dry sand under transient loading conditions was illustrated by test data from studies at Harvard University *(Ref. 1)* and M.I.T. *(Ref. 2)* and the mechanics of strength mobilization in saturated sands under transient loading by investigations conducted at the University of California *(Ref. 3)*.

A somewhat more detailed discussion of the results of repeated loading studies was then presented with particular reference to their application in pavement design. Recent studies conducted by the California Division of Highways *(Ref. 4)* have shown that in the case of pavements not only must the cumulative pavement deformation be considered but also the possibility of fatigue failures in the surfacing resulting from excessive resilient deformations.

Even in cases where pavements have shown no appreciable permanent deformation, failures of the surfacing have been observed. By making measurements of resilient deformation of pavements at a number of locations throughout the State it has been found that such failures are associated with large numbers of stress applications and the more resilient subgrade soils. The implication of these results was discussed.

Investigations of the plastic deformation of compacted clays under repeated loading were then described. Apparatus used for testing soils under these conditions was illustrated *(Ref. 5)* and data were presented to demonstrate the influence of frequency of stress application and stress history on the results obtained *(Refs. 6, 7)*. A brief discussion of thixotropy in soils *(Ref. 8)* and its influence in repeated load studies was included in this discussion.

The significant effects that a previous stress history in the form of a series of repeated stress applications may have on the subsequent deformation of a soil was illustrated by a comparison of data from test series in which specimens were subjected to a progressive increase in repeated axial stress and in which previously unloaded specimens were subjected to similar stress intensities.

It was shown that an entire sequence of 240,000 stress applications (consisting of 30,000 repetitions at each of eight different stress intensities) caused only as much deflection as would the last 1,500 applications of the series applied alone. In the light of these results the practical difficulty of assessing the cumulative effects of a series of stress applications of different magnitudes from data concerning their individual effects was emphasized.

Finally, studies of soil strength under earthquake loading conditions were described. The results of model tests (scale 1:150) conducted at the University of California *(Ref. 9)* to investigate the earthquake resistance of rockfill dams were presented and used to show that applied shocks produced no significant effects (appreciable changes of section) on the models until the progressively increasing accelerations exceeded 0.4 g; also that even when the accelerations of the test earthquakes were increased to more than 1 g the models suffered only small changes of shape without any major failure developing.

More recent tests on compacted clay under simulated earthquake loading conditions were also briefly described.

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1. Casagrande, A. and Shannon, W. L. "Research on Stress-Deformation and Strength Characteristics of Soils and Soft Rocks under Transient Loading," Publication, Graduate School of Engi-

neering, Soil Mechanics Series No. 31, Harvard University, 1948.

2. Whitman, R. V. "The Behavior of Soils under Transient Loading," Proceedings, Fourth International Conference on Soil Mechanics and Foundation Engineering, 1957.
3. Seed, H. B. and Lundgren, R. "Investigation of the Effect of Transient Loading on the Strength and Deformation Characteristics of Saturated Sands," Proceedings, American Society for Testing Materials, Vol. 54, 1954.
4. Hveem, F. N. "Pavement Deflections and Fatigue Failures" in Design and Testing of Flexible Pavement, HRB Bulletin 114.
5. Seed, H. B. and Fead, J. W. N. "Apparatus for Repeated Load Tests on Soils," Paper presented at Annual Meeting American Society for Testing Materials, Atlantic City, June, 1959.
6. Seed, H. B., McNeill, R. L. and deGuenen, J. "Increased Resistance to Deformation of Clay Caused by Repeated Loading," Proceedings, American Society of Civil Engineers, Paper No. 1645, May, 1958.
7. Seed, H. B. and Chan, C. K. "Influence of Stress History and Frequency of Stress Application on the Deformation of Highway Subgrades under Repeated Loading," Proceedings, Highway Research Board, Vol. 37, 1958.
8. Seed, H. B. and Chan, C. K. "Thixotropic Characteristics of Compacted Clays," Transactions, American Society of Civil Engineers, 1958.
9. Clough, R. W. and Pirtz, D. "Earthquake Resistance of Rock Fill Dams," Journal, Soil Mechanics and Foundations Division, American Society of Civil Engineers, No. SM2, April, 1956.

Foundations for Large Bridges Across San Francisco Bay

H. Bolton Seed

San Francisco Bay, an area of 463 square miles of water connected to the Pacific Ocean by a narrow gorge about one mile wide, has always presented a challenge to the civil engineer. Crossing the Bay and forming a part of the California highway system are four major bridge structures: the San Francisco-Oakland Bay Bridge, the Golden Gate Bridge, the Richmond-San Raphael Bridge and the Carquinez Bridge. The paper outlined the type of foundations used in the construction of these bridges.

The San Francisco-Oakland Bay Bridge, opened in 1936, has a length of about seven miles, including ramps and fills, and spans about 4½ miles across the water. The methods of construction of pier foundations included the use of steel sheet pile cofferdams, Moran pneumatic caissons *(Ref. 1)* sunk in some cases to depths of 240 ft and open caissons *(Ref. 2)*. The techniques of sinking the Moran pneumatic caissons were illustrated and discussed.

The Golden Gate Bridge, spanning across one mile of water, was completed in 1937. The long suspension span of 4,200 ft resulted in only two piers being constructed under water. Difficulties encountered in the construction of an access trestle 1,100 ft long, a concrete fender wall and a pneumatic caisson, finally resulting in the use of the fender as a cofferdam for pier construction, were described *(Ref. 3)*.

The Richmond-San Raphael Bridge, spanning four miles from Richmond to San Raphael, was completed in 1947. Of the 79 piers for the bridge, nine were built on land, eight were built in cofferdams in the shallow waters near the eastern bridge terminus and the remaining 62 were of the bell bottom type for which the contractor elected to use precast concrete structural units. The sequence of construction operations for the bell bottom type piers was reviewed and illustrated *(Refs. 4, 5)*.

The most recently completed of the Bay bridges is the Carquinez Bridge, spanning 3,350 ft across the Bay and continuing to a total length of about 5,300 ft *(Refs. 6, 7)*. The foundations for the Bay portion of the bridge involved the construction of an anchorage abutment set in a shale and sandstone cliff at the north end of the bridge, three caisson piers founded on bedrock about 132 ft below the water line, a cofferdam type pier supported on 240 steel bearing piles and an anchorage pier 125 ft high located at the south end of the bridge. Caisson construction involved the novel use of relatively thin precast concrete slabs for the construction of caisson walls, thus decreasing the weight of the caisson and eliminating the need for outside forms.

It was hoped that the variety of techniques involving the use of long-established procedures, new ideas and the incorporation of modern developments which have been used for the construction of bridge foundations across San Francisco Bay would provide some idea of typical U. S. practice in the construction of foundations for large bridges.

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3. The Golden Gate Bridge—Report of the Chief Engineer to the Board of Directors of the Golden Gate Bridge and Highway District, California, January, 1938.
4. Raab, N. A. "Pier Construction," California Highways and Public Works, March-April, 1954, Vol. 33, Nos. 3-4.
5. Gerwick, B. C. "Hollow Precast Concrete Units of Great Size Form Bridge Substructure," Civil Engineering, Vol. 24, No. 4, April, 1954.
6. Hollister, L. C. "Carquinez Bridge," California Highways and Public Works, September-October, 1956, Vol. 35, Nos. 9-10.
7. Hollister, L. C. "New Parallel Bridge," California Highways and Public Works, January-February, 1959.

Appendix

List of Russian Language Publications

Members of the delegation received a number of publications from various persons and agencies during their visit in the Soviet Union. These have been filed with the Highway Research Board in Washington. The list of these titles follows:

1. "Investigations of Frost Heaving of Highways," Edited by N. V. Ornatsky. Moscow, 1941. 428 pp. A series of articles concerning observations of frost boils, laboratory investigations, and experimental designs of preventive measures.
2. Ter-Stepanian, G. I. "Effect of Particle Arrangement on the Shear Process in Soils." Acad. of Sciences of Armenian SSR, Erivan, 1943.
3. Ter-Stepanian, G. I. "On the Accumulative Ridges." Acad. of Sciences of Armenian SSR, Erivan, 1946. 3 pp.
4. Goldstein, M. N. "Deformation of Roadbed and Foundation of Structures During Freezing and Thawing." Inst. of Railroad Transport. Moscow, 1948. 211 pp.
5. Beroulia, A. K., Beroulia, V. I. and Nosich, I. A. "Stability of Roadbed Soils in Steppe Regions." Moscow, 1951, 176 pp.
6. Beroulia, A. K., et al. A Symposium. Publications of the Kharkov Auto-Roads Institute, Kharkov, 1952, No. 13. 186 pp. Twelve articles.
7. A Symposium. Publications of the Kharkov Auto-Road Institute, Kharkov, 1953, No. 14, 127 pp. Eleven articles.
8. Litvinov, I. M. "Soil Investigations under Field Conditions." 2nd Edition, Moscow, 1954. 223 pp.
9. "Program for Course in Soil Mechanics and Foundations," (prepared for students in hydraulic structures). Moscow, 1955. 8 pp.
10. "Program of Study for Course in Foundations." Dept. of Mines, Moscow, 1955. 8 pp.
11. "Instructions for the Determination of the Required Density and Compaction Control of Road Embankments." Ministry of Auto Transport and Roads USSR, Moscow, 1955. 32 pp.
12. Beroulia, A. K. "Roads of Local Materials." Moscow, 1955. 139 pp.
13. Vialov, S. S. "Relation between Stress and Strain of Frozen Grounds under Consideration of the Factor Time." Acad. of Sciences, Moscow, 1956. 4 pp.
14. "Conference Reports on Theory of Design of Beams and Slabs on Compressible Bases (No. 14)." Moscow, 1956. 238 pp.
15. Vialov, S. S. and Tsytovich, N. A. "Estimation of the Bearing Capacity of Cohesive Soils from the Penetration of a Spherical Plunger." Acad. of Sciences, Moscow, 1956. 4 pp.
16. Dalmatov, B. I. "Effect of Frost Heaving of Soils on Foundations of Structures," Leningrad, Moscow, 1957. 59 pp.
17. Ter-Stepanian, G. I. "Investigation of Subsurface Sliding of Slopes." Acad. of Sciences of Armenian SSR, Erivan, 1957. 13 pp.
18. Askalanov, V. V. and Tokin, A. N. "Buildings and Structures from Soil-Cement." Moscow, 1957. 111 pp.
19. Pukavtsov, A. M. and Perley, E. M. "Use of Vibratory Pile Drivers in Industrial Construction." Leningrad, Moscow, 1957. 78 pp.
20. Lukyanov, U. S. and Golovko, M. D. "Analysis of Depth of Soil Freezing." Institute of Transportation Engineering, Moscow, 1957. 165 pp.
21. Beroulia, A. K. and Sidenko, V. M. "Determination of Design Water Contents of the Roadbeds of Autoroads on the Basis of the Theory of Probability," Kharkov Auto-Road Institute, Kharkov, 1958. 36 pp.

22. Beroulia, A. K. "Compaction of Soils and of Road Pavements by Pneumatic-Tired Rollers," Kharkov Auto-Road Institute, Kharkov, 1958. 23 pp.
23. A Symposium Publications of the Kharkov Auto-Road Institute, Kharkov, 1958, No. 21, 184 pp. Seventeen articles.
24. Vastchenko, B. I. "Study of Mixing Soil with Binder in Road Pavement Construction," Kharkov Auto-Road Institute, Kharkov, 1958. 39 pp.
25. "Bulletin of Theoretical and Technical Information." Published by "Hydroproject" (Institute for design of hydropower plants). Moscow No. 4, 1958. 76 pp. A series of articles on planning and design, investigations, experimental studies, and on actual construction operations of hydroelectric projects.
26. "Bulletin of Theoretical and Technical Information." Published by "Hydroproject" (Institute for design of hydropower plants). Moscow No. 3, 1958. 123 pp. A series of articles pertaining to various aspects of Lenin and Kuibyshev power plants.
27. Vialov, S. S. "Third Group of Antarctic Explorers." Acad. Sciences, Moscow, 1958. 6 pp.
28. Vialov, S. S. "Lasting Strength of Frozen Soils and Their Bearing Capacity." Acad. of Sciences, Moscow, 1958. 2 pp.
29. "Aspects of Soil Mechanics." Leningrad Civil Engineering Institute. Leningrad, 1958. 203 pp. (A compilation of 10 articles).
30. Ter-Stepanian, G. I. "Classification of Landslide Cracks." Acad. of Sciences of Armenian SSR, Erivan, 1958. 16 pp.
31. "Problems of Design of Slabs on Elastic Foundations." Edited by B. G. Korenev. Acad. of Construction and Architecture, Moscow, 1958. 120 pp. Three articles.
32. Goldstein, M. N. "Generalization of the Equations of the Thermodynamics of Nonreversible Processes." Acad. of Sciences, Moscow, 1958.
33. "Planning and Structural Development of Inhabited Regions of the Far North." Leningrad, 1958. 122 pp.
34. Ornatsky, N. V. "On the Deformability of Water Saturated Soils," 1958. 5 pp.
35. Vialov, S. S. "Regularities of Glacial Shields Movement and the Theory of Plastic Viscous Flow" (In English). 1958. 10 pp.
36. Vialov, S. S. "Regularities of Ice Deformation" (In English). 1958. 9 pp.
37. "A World Record of Sinking Shaft for Mine Nova-Boutirka." Moscow, 1959. 12 pp.
38. "Reports from the USSR to the XIth International Road Congress." Moscow, 1959. 198 pp.
39. A Symposium. Publications of the Kharkov Auto-Road Institute, Kharkov, No. 20, 1959. 183 pp. Twenty articles.
40. "Instructions in the Use of Precast Blocks in Bearing Walls." Committee on Construction of the Cabinet of Ministers. Moscow, 1959. 30 pp.
41. Theoretical and Technical Bulletin "Bases and Foundations." Acad. of Construction and Architecture. Moscow, 1959. No. 22. 40 pp. A series of articles including such subjects as piles, soil pressures on retaining structures, electro-osmotic characteristics of soils, etc.
42. "Automobile Highways" (a magazine). A publication of the Dept. of Construction of Automobile Highways. Moscow. Vol. 12, No. 1 through 9, incl., 1959. Each issue contains articles on various aspects of highway design, construction, maintenance, finance, construction equipment, etc.
43. Proceedings of "Hydroproject" (Institute for design of hydro-power plants). No. 2, 1959. 177 pp. Part I is a series of articles on hydraulic investigations and Part II is analysis and investigation of construction aspects of hydraulic structures.
44. "Bulletin of Theoretical and Technical Information." Published by "Hydroproject." Moscow No. 5, 1959. 88 pp. A series of articles on design and construction of the Stalingrad hydroelectric system, and also articles on re-

lated problems including one on shear resistance of clay.

45. "Artificial Foundations of Structures." Acad. of Construction and Architecture. Moscow, 1959. 68 pp. Some of the six articles concern artificial freezing of soils.
46. "Problems of Construction on Macroporous Settling Soils." Acad. of Construction and Architecture. Moscow, 1959. 75 pp. A series of five articles.
47. Gorbounov-Possadov, M. I. "Tables for the Design of Thin Slabs on Elastic Foundations." Moscow, 1959. 98 pp.
48. Ukhov, S. B. "Artificial Salting of Clay Soils for Winter Construction." Novosibirsk, 1959. 13 pp.
49. "Reports of Conference on Rational Methods of Foundation Construction on Permafrost." Acad. of Construction and Architecture. Moscow, 1959. 132 pp. A series of 17 articles.
50. Tatarnikov, B. P. "Low-Frequency (Resonance) Vibrator for Driving Thin - Shelled Reinforced Concrete Cylinders." Leningrad Inst. for Railroad Engineers. Leningrad, 1959. 18 pp.
51. Goldstein, M. N. and Gutman, E. M. "Effect of High Frequency Ultrasonic Field on Plastic Paste." Acad. of Sciences. Moscow, 1959. 4 pp.
52. "Instructions on Surface Compaction of Soil in the Foundations of Buildings and Industrial Structures by Means of Heavy Rammers." Committee on Construction of the Council of Ministers. Moscow, 1959. 30 pp.
53. "Instructions for Depth Compaction of Macroporous or Loess Soils (Subject to Settling) by Means of Piles in the Foundations of Buildings and Structures." Committee on Construction of the Council of Ministers. Moscow, 1959. 36 pp. (A manual and specifications on such construction.)
54. "Technical Conditions for Design and Construction of Earth Fill Dams from Dense Clay." Ministry of Construction. Leningrad, 1959. 45 pp. (A manual and specifications on such construction.)
55. Askalonov, V. V. "Stabilization by Silicates of Loess Soils." Moscow, 1959. 78 pp.
56. Turovskaya, A. Ya. "Investigation of Strength of Primary Kaolin." Dept. of Transportation, Dnepropetrovsk, 1959. 20 pp.
57. Baranov, D. S. "Measuring Instruments and Methods and Some Results of Investigation of Pressure Distribution in Sandy Soil." Acad. of Constr. and Arch. Moscow, 1959. 62 pp.
58. "Principles of Geocryology (Frost Science) Vol. I: General Geocryology." Acad. of Sciences. Moscow, 1959. 459 pp. Contains 13 chapters prepared cooperatively by many workers in this field.
59. "Principles of Geocryology (Frost Science), Vol. II: Engineering Geocryology." Acad. of Sciences. Moscow, 1959. 365 pp. Contains 13 chapters.
60. Sergeyev, E. M. "Soil Science." Moscow University Press, 1959. 334 pp.
61. Drannikov, A. M. "Engineering Geology." Gosstroyizdat, Ukrainian SSR. Kiev, 1959. 223 pp.
62. Mkhitaryan, A. M. "Hydraulics and Principles of Gas Dynamics." Gostekhizdat. Kiev, 1959. 279 pp.
63. "Laboratory Manual on Soil Mechanics." Issued by Moscow Civil Engineering Institute. 20 pp.

The Soviet Delegation, at the time of its visit to Washington in June 1959, presented a number of publications to the Highway Research Board. These are also on file. This list of titles follows:

1. "Instructions on the Strengthening and Reconstruction of Reinforced Concrete Structures by the Method of Engineer I. M. Litvinov." (Youzh. N. I. I.) Kharkov, 1948. 38 pp.
2. Gorbounov-Possadov, M. I. and Krechmer, V. V. "Graphs for Stability Computations of Foundations." State Publishing House of Literature on Construction, Architecture and Construction Materials. Moscow 1951. 56 pp.

3. Litvinov, I. M. "Apparatus for the Testing of Soils in Shear Under Field Conditions." (Youzh. N.I.I.) Published by Academy of Architecture, Ukrainian SSR. Kiev, 1954. 34 pp.
4. Litvinov, I. M. "Equipment for Soil Studies Under Field Conditions." (Youzh. N. I. I.) Published by Academy of Architecture, Ukrainian SSR. Kiev, 1954. 28 pp.
5. Litvinov, I. M. "Apparatus for Accelerated Determination of Basic Physical Characteristics of Soils for the Control of the Quality of Production of Earthworks." (Youzh. N. I. I.) Published by Academy of Architecture, Ukrainian SSR. Kiev, 1954. 23 pp.
6. "Drying Oven for Field Investigations." (Youzh. N. I. I.) Published by Academy of Architecture, Ukrainian SSR. Kiev, 1954. 26 pp.
7. "Gages System I. M. Litvinov for the Measurement of Cracks in Building Structures." (Youzh. N. I. I.) Southern Scientific-Research Institute on Construction. Published by Academy of Architecture, Ukrainian SSR. Kiev, 1954. 10 pp.
8. Litvinov, I. M. "Soil Investigations Under Field Conditions." Second Edition—Amplified. Ougletehizdat. Moscow, 1954. 223 pp.
9. Vialov, S. S. and Tsytovich, N. A. "Reports (DOKLADY) Academy of Sciences, USSR, 1955. Vol. 104—No. 4." Reprint of pages 527 to 529, paper: "Cohesion of Frozen Soils."
10. "Deformations of Soils at Freezing and Thawing." Publication No. 26 (NIIOPS) State Publishing House of Literature on Construction, Architecture and Construction Materials. Moscow, 1955. 95 pp.
11. Litvinov, I. M. "Thermic Stabilization of Collapsible Loess and Other Soils in the Bases of Various Buildings and Structures." Published by Academy of Architecture, Ukrainian SSR. Kiev, 1955. 55 pp.
12. Vialov, S. S. and Tsytovich, N. A. "Reports (DOKLADY) Academy of Sciences USSR 1956. Vol. 111—No. 6." Pages 1193 to 1196, paper: "Estimation of the Bearing Capacity of Cohesive Soils from the Penetration of a Spherical Plunger."
13. Tsytovich, N. A. "Reports (DOKLADY) Academy of Sciences, USSR 1956. Vol. 111—No. 5." Reprint of pages 965 to 968, Paper: "On the Determination of the Forces of Adhesion of Cohesive Soils by the Ball Test Method."
14. Vialov, S. S. "Reports (DOKLADY) Academy of Sciences USSR 1956. Vol. 108—No. 6." Reprint of pages 149-152, Paper: "Relationship Between Stress and Strain of Frozen Grounds under Consideration of the Factor 'Time'."
15. "Instructions for Investigations of Construction Properties of Soils in Field Laboratories System of I. M. Litninov." State Publishing House of Literature on Construction and Architecture. Moscow, 1956. 54 pp.
16. "Instructions on the Thermic Stabilization of Collapsible Microporous (Loess) Soils." State Publishing House of Literature on Construction and Architecture. Moscow, 1956. 31 pp.
17. Bezruk, V. M. "Theoretical Basis of Soil Stabilization by Cements." (SOYUZDORNII) Scientific - Technical Publishing House of Autotransport Literature. Moscow, 1956. 248 pp.
18. Tsytovich, N. A. Reprint of Article (in English) "The Fundamentals of Frozen Ground Mechanics (New Investigations)." Proceedings, Fourth International Conference on Soil Mechanics and Foundation Engineering, London, 1957.
19. "Instructions for the Selection of Road Flexible Pavement Designs." (GLAVDORSTROY) Scientific and Technical Publishing House of Autotransport Literature. Moscow, 1957. 71 pp.
20. Khalizev, E. P. "Selection of the Optimum Regime for Work of Hydro-Mechanical Installations in Caissons." (NIIOPS) State Publishing House of Literature on Construction, Architecture, and Construction Materials. Moscow, 1957. 52 pp.

21. Harhouta, N. I. and Vassiliev, U. M. "Deformations of Soils in Road Embankment Fills." (SOYUZDORNII) Scientific-Technical Publishing House of Autotransport Literature. Moscow, 1957. 73 pp.
22. Ivanov, N. N. "Construction of Automobile Roads—Part II: Construction of Road Pavements." Scientific-Technical Publishing House of Autotransport Literature. Moscow, 1957. 338 pp.
23. "Data on Laboratory Studies of Frozen Soils." Publication No. 3 Academy of Sciences, USSR. Moscow, 1957. 324 pp.
24. Tsytovich, N. A. "On the Design of Foundations for Limit Conditions of Soil Bases." Reprint from book of scientific reports of the Czech Polytechnical Institute in honor of the 250th Anniversary. Prague, 1958 (in Russian).
25. Oushkalov, B. P. "Data Concerning Fundamentals of the Science of Frozen Zones of the Earth's Crust." Published by Academy of Sciences USSR. Moscow, 1958. Reprint pages 50-87 inclusive, article, "Depth of Thawing of Frozen Soils Under Structures and Its Limiting Permissible Values."
26. "Problems of Investigation of Soil-Bases of Structures." Publication No. 33. (NIIOPS) State Publishing House of Literature on Construction, Architecture and Construction Materials. Moscow, 1958. 100 pp.
27. "Soil Mechanics." Publication No. 34. (NIIOPS) State Publishing House of Literature on Construction, Architecture and Construction Materials. Moscow, 1958. 123 pp.
28. Efremov, M. G. "Vibro Method of Sinking Geological Reconnaissance Bore Holes." (NIIOPS) State Publishing House of Literature on Construction, Architecture and Construction Materials. Moscow, 1958. 43 pp.
29. "Instructions for the Determination of the Relative Compression of Frozen Soils When Thawed Under Pressure." (NIIOPS) State Publishing House of Literature on Construction, Architecture and Construction Materials. Moscow, 1958. 15 pp.
30. "Construction of Road Bases and Pavements Over Saline Soils and Gravel Materials Treated by Bitumens and Tars." (SOYUZDORNII) Scientific and Technical Publishing House of Autotransport Literature. Moscow, 1958. 208 pp.
31. "Instructions on the Vibro Method of Sinking Geological Exploratory Bore Holes for Construction Purposes." State Publishing House of Literature on Construction and Architecture. Moscow, 1958. 52 pp.
32. "Ground Water Level Lowering by Light Wellpoint Installations and by Ejector Wellpoints." (Construction Manual) State Publishing House of Literature on Construction and Architecture. Moscow, 1958. 110 pp.
33. "Temporary Technical Instructions for the Constructions of Tunnels by the Shield Method." State Publishing House of Literature on Construction and Architecture. Moscow, 1958. 180 pp.
34. "Instructions for the Determination of the Design Depth of Thawing of Frozen Soils in the Bases of Structures and on the Determination of Design Thermo-Physical Coefficients of Soils." (NIIOPS) State Publishing House of Literature on Construction, Architecture and Construction Materials. Moscow, 1958. 18 pp.
35. "Temporary Instructions on the Dewatering of Construction Cofferdams in Clayey Soils by the Use of Wellpoint Stabilization and Direct Electrical Current." (NIIOPS) State Publishing House of Literature on Construction, Architecture and Construction Materials. Moscow 1958. 16 pp.
36. "Instructions on the Use in Road and Airfield Construction of Soils Stabilized by Binding Materials." S.N.—25-58. State Publishing House of Literature on Construction, Architecture and Construction Materials. Moscow, 1958. 69 pp.
37. Silin, K. S., Glotov, N. M., Gretsov, A. E., Kappinsky, V. I. and Prohorov, A. D. "Bridge Pier Foundations of

Composite Reinforced Concrete Cylinders." State Transport Publishing House. Moscow, 1958. 200 pp.
38. "Bases, Foundations, Soil Mechanics." Scientific-Technical Journal of the State Committee of the Council of Ministers USSR on Construction Methods. No. 1—1959 (First issue). Moscow. 32 pp. *Note:* Contains several articles and (pages 27-28) gives the membership of the technical committees of the Commission (National Association USSR) on Soil Mechanics and Foundation Engineering KOMGF.
39. Same publication. No. 2—1959. 33 pp.
40. Same publication. No. 3—1959. 32 pp. (Most of the issue is devoted to articles on permafrost construction.)
41. "Transport Construction." Journal of the Ministry of Transport Construction. No. 1. January 1959 (Ninth year of publication). Moscow. 65 pp.
42. Same publication. No. 3—March 1959. 65 pp.
43. Same publication. No. 4—April 1959. 65 pp.
44. "Journal of the Academy of Construction and Architecture, Ukrainian SSR" (in Ukrainian language). No. 1—1959. Kiev, 1959. 98 pp. *Note:* Contains two page summaries of the articles of the issue in the Chinese, English, French and German languages. Also 3-page list (in Ukrainian) of all members of the Academy and of its dependent organizations.
45. Litvinov, I. M. "Basic Requirements for the Design and Performance of Work on Thermic Soil Stabilization." Published by Academy of Construction and Architecture, Ukrainian SSR. Kiev, 1959. 54 pp.
46. Korsounsky, M. B. "Inspection and Evaluation of the Strength of Roads with Flexible Pavements." (SOYUZDORNII) Scientific-Technical Publishing House of the Ministry of Automobile Transport and Paved Roads, RSFSR. Moscow, 1959. 64 pp.
47. "Instructions on the Organization and Performance of Observations Related to Changes in the Water Temperature Regime of Permanently Frozen Grounds for Purposes of Foundation Construction." State Publishing House of Literature on Construction, Architecture and Construction Materials. Moscow, 1959. 27 pp.
48. Vinokourov, F. P., Teterkin, A. E. and Piterman, M. A. "Peat in Construction Work." Published by Academy of Sciences BSSR, Minsk, 1959. 241 pp.
49. Ornatsky, Prof. N. V. (General Editor). "Studies of Strength of Road Pavements." (SOYUZDORNII) Scientific-Technical Publishing House of the Ministry of Automobile Transport and of Paved Roads RSFSR. Moscow, 1959. 200 pp.
50. "Reports from USSR to the XIth International Road Transport Congress." (Rio de Janeiro 1959) Scientific-Technical Publishing House of the Ministry of Automobile Transport and of Paved Roads of the RSFSR. Moscow, 1959. 128 pp.
51. Vialov, S. S. and Egorov, N. I. "Book of Articles on Engineering Permafrost Studies." Vol. XIV Academy of Sciences USSR. Reprint pages 40-55 inclusive, paper "Experimental Determination of Frost Heaving of Soils."

Dr. Tschebotarioff, a member of the delegation, has also acquired a large number of Russian language publications on soil mechanics and foundation engineering. Some, but not all, of these were obtained on the exchange visit. These publications have been loaned by Dr. Tschebotarioff to the Library of the River and Harbor Section, Department of Civil Engineering, Princeton University, Princeton, New Jersey. That library can supply microfilms or photostats thereof at cost to interested persons or send originals on usual terms for inter-library loans.

1. Pouzirevskaya, T. N. "Seepage of Water Through Sandy Soils." Transactions of the Scientific Research Institute of Hydrotechnics, Vol. I, Oct. 1931. Pp. 251-256 (photostat).
2. Tsytovich, N. A. "Fundamentals of Soil Mechanics." Leningrad, 1934. 307 pp.
3. Kisselyov, V. A. "Beams and Frames on an Elastic Base." Moscow, 1936. 228 pp. (Photostat of Table of Contents and of pp. 13-27.)
4. Gersevanov, N. M. "Fundamentals of the Dynamics of a Soil Mass." Moscow, 1937. 242 pp., 3rd edition.
5. Florin, V. A. "Determination of Instantaneous Stresses in the Skeleton of a Soil Mass." Doklady, Academy of Sciences, USSR, 1937, Vol. XVI, No. 8. Pp. 395-398 (photostat).
6. Florin, V. A. "On the Problem of Hydrodynamic Stresses in a Soil Mass." Pamphlet of Glavgidroenergostroy. Leningrad, 1938. 54 pp.
7. Abeleff, Y. M. et al. Symposium: "Construction on Loessial Soils." Moscow, 1939. 288 pp.
8. Ornatsky, N. V. et al. Symposium: "Studies of Frost Boils on Automobile Roads." Moscow, DORNII, 1941. 428 pp.
9. Deryagin, B. V. and Krylov, N. A. "Anomalous Phenomena During Flow of Liquids Through Rigid Filters with Narrow Pores." Academy of Sciences, USSR, Institute of Machine Studies. 2 pp. (photostat).
10. Denissov, N. J. and Rehbinder, P. A. "On the Colloido-Chemical Nature of the Cohesion in Argillaceous Rocks." Doklady, Academy of Sciences USSR, 1946, Vol. LIV, No. 6. Pp. 523-526 (photostat).
11. Ditto, English language Reprint (marked pp. 519-522).
12. Denissov, N. J. "On the Stage-Wise Deformations of Argillaceous Rocks." Doklady, Academy of Sciences USSR, Vol. LVI, No. 1, 1947. Pp. 73-76 (English language Reprint).
13. Reltov, B. F. "Settlement Phenomena in Cohesive Water-Saturated Soils." Transactions of the Scientific Research Institute of Hydrotechnics, Vol. 33, 1947. Pp. 5-23 (photostat).
14. Florin, V. A. "Consolidation Problems of Soil Masses." Transactions of the Scientific Research Institute of Hydrotechnics, Vol. 34, 1947. Pp. 133-146 (reprint).
15. YOUZHNII* "Instructions for the Strengthening and Reconstruction of Reinforced Concrete, Structures by the Method of Engineer I. M. Litvinov." Kharkov, 1948. Pamphlet, 38 pp.
16. Doubelir, G. D. and Korneyev. "Construction of Roadbeds in Slide and Rockfall Areas." DORIZDAT, Moscow, 1948. 137 pp.
17. Florin, V. A. "Some Simplest Nonlinear Consolidation Problems of a Water-Saturated Soil Medium." Izvestia, Academy of Sciences USSR, Section of Technical Sciences, No. 9, 1948. Pp. 1389-1402 (reprint).
18. Gorbounov-Possadov, M. I. "Beams and Slabs on an Elastic Base." Mashstroyizdat, Moscow, 1949. 233 pp.
19. Schelkachov, V. N. and Lapouk, B. B. "Underground Hydraulics," Edited by Academician L. S. Leibensohn. Moscow, 1949. 523 pp.
20. Byelyi, L. D. et al. Symposium: "Engineering-Geological Studies for Hydro-Power Construction." Gossgeolizdat, Moscow, 1950, Vol. I. 320 pp.
21. Ditto, Vol. II. 354 pp.
22. Ornatski, N. V. "Soil Mechanics." (A Textbook for Geologists) Moscow University Publication, 1950. 420 pp.
23. Pilgounova, Z. V. "Method of Mathematical Statistics for Determination of Average Shearing Resistance Value of Clay Soils." (Publication not given.) Pp. 233-242 (reprint).
24. Goldstein, M. N. "Model Studies of Deformation of Base during Freezing." (Publication not given.) Pp. 296-311 (reprint).
25. Rosa, S. A. "Settlements of Hydrotechnical Structures on Clays of Low Humidity." Gidrotechnicheskoye

* YOUZHNII is the abbreviation for: "Southern Scientific Research Construction Institute."

Stroitelstvo, No. 9, 1950. Pp. 25-30 (photostat).

26. Goloubkov, V. N. "Bearing Capacity of Pile Foundations." Mashstroyizdat, Moscow, 1950. 142 pp.
27. Florin, V. A. "Consolidation of Earth Media and Seepage under Conditions of Variable Porosity and Consideration of the Influence of Bound Water." Izvestia, Academy of Sciences USSR, Section of Technical Sciences, 1951, No. 11. Pp. 1625-1649 (reprint).
28. Gorbounov-Possadov, M. I. and Krechmer, V. V. "Graphs for the Design of Foundation Stability." Moscow, 1951. 56 pp.
29. Denissov, N. J. "Construction Properties of Loess and of Loessial Loams." Moscow, 1953. 154 pp.
30. Florin, V. A. "One-Dimensional Consolidation Problem of Compressible Creeping Earth Media." Izvestia, Academy of Sciences USSR, Section of Technical Sciences, 1953, No. 6. Pp. 797-812 (reprint).
31. Florin, V. A. "One-Dimensional Consolidation Problem under Consideration of Aging, of Non-Linear Creep and of Destruction of Structure." Izvestia, Academy of Sciences USSR, 1953, No. 9. Pp. 1229-1234 (reprint).
32. Goldstein, M. N. et al. "Problems of Geotechnics." Moscow, 1953. 228 pp.
33. Ouginchouss, A. A. "Canals and Structures Thereon." Moscow, 1953. 392 pp.
34. Gorbounov-Possadov, M. I. "Design of Structures on an Elastic Base." Gosstroyizdat. Moscow, 1953. 516 pp.
35. Litvinov, I. M. "Soil Investigations under Field Conditions," 2nd edition. Ougletechizdat. Moscow, 1954. 223 pp.
36. Litvinov, I. M. "Apparatus for Soil Investigations under Field Conditions." YOUZHNII. Kiev, 1954. 28 pp.
37. Litvinov, I. M. "Apparatus for Accelerated Determination of Fundamental Physical Properties of Soils during Quality Control of Earth-works." YOUZHNII. Kiev, 1954. 23 pp.
38. Litvinov, I. M. "Apparatus for Shear Testing of Soils under Field Conditions." YOUZHNII. Kiev, 1954. 33 pp.
39. Litvinov, I. M. "Drying Oven for Field Investigations." YOUZHNII. Kiev, 1954.
40. YOUZHNII. "Gages System I. M. Litvinov for the Measurement of Cracks in Structures." Kiev, 1954. 9 pp.
41. Ouginchouss, A. A. and Bombchinsky, V. P. "Equipment for Control Measurements on Hydrotechnical Structures." Moscow, 1954. 259 pp.
42. Popov, I. V. "Summary of Conference on Engineering-Geological Studies of Soil Deposits." (Publication not given.) 1954. 3 pp. (reprint).
43. Litvinov, I. M. "Thermic Stabilization of Settling Loess and other Soils in the Bases of Buildings and Structures." YOUZHNII. Kiev, 1955, 55 pp.
44. Ornatsky, N. V., Sergeyev, E. M. and Shechtman, Y. M. "Study of the Process of Colmatation of Sands." Moscow University, 1955. 182 pp.
45. Barkan, D. P. et al. A Symposium: "Conference to Report on Scientific Research Work in 1954." (Summary of papers and reports presented.) Scientific Research Institute on Foundations, Moscow, 1955.
46. "Rules and Technical Specifications for the Design of Natural Bases of Buildings and Industrial Structures." State Construction Committee of the Council of Ministers USSR. Moscow, 1955.
47. "Instructions for the Determination of the Required Density and Compaction Control of Road Embankments." Ministry of Auto Transport and Roads USSR. Moscow, 1955. 32 pp. (photostat).
48. Vassiliev, B. D. "Bases and Foundations." Leningrad, 1955. 383 pp.
49. Masslov, N. N. "Stability Conditions of Slopes in Hydro-Power Construction." Gossenergoizdat. Moscow, 1955. 468 pp.
50. Savinov, O. A. "Machine Foundations." Leningrad, 1955. 292 pp.

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52. Barkan, D. D. et al. A Symposium (No. 27): "Soil Dynamics." The Scientific-Research Institute on Foundations. Moscow, 1955. 80 pp.
53. Popov, I. V. "The Role and Conditions for the Use of Mechanics in Geology." Report at the Scientific Anniversary Session of the Moscow State University in 1955. Pp. 5-14 (reprint).
54. "Program for Course on Bases and Foundations" (for Civil Engineering students specializing in industrial and housing construction). Moscow, 1955. 8 pp.
55. "Program for Course on Soil Mechanics, Bases and Foundations" (for Civil Engineering students specializing in hydrotechnical and waterfront structures). Moscow, 1955. 8 pp.
56. "Notebook for Laboratory Work of the Course on Bases and Foundations." Moscow Civil Engineering Institute, 195-. 20 pp.
57. Denissov, N. J. "Construction Properties of Clay Soils and Their Utilization in Hydrotechnical Construction." Gossenergoizdat. Moscow, 1956. 288 pp.
58. Tokar, R. A. et al. A Symposium (No. 30): "Soil Mechanics." Scientific Research Institute on Foundations. Moscow, 1956.
59. Popov, I. V. "Theoretical Basis for Engineering-Geological Classification of Soil and Rock Deposits." (Publication not given.) Pp. 198-215 (reprint).
60. Stomahin, I. B. "Main Lines of Improvement in Design Work for Construction." Gosstroyizdat. Moscow, 1956. 35 pp.
61. MINSTROY* "Instructions for Deep Consolidation of Weak Water-Saturated Soils by Means of Sand Piles when Arranging Bases for Buildings and Structures." Moscow, 1956. 46 pp.

* MINSTROY is the abbreviation for "Ministry of Construction."

62. "Instructions for the Investigation of Construction Properties of Soils by the Field Laboratory, System I. M. Litvinov." Moscow, 1956. Construction Ministry of the Metallurgical and Chemical Industry USSR. 55 pp.
63. "Instructions for the Thermic Stabilization of Settling Macroporous (Loessial) Soils." Moscow, 1956. Construction Ministry of the Metallurgical and Chemical Industry USSR. 31 pp.
64. Babkov, V. F. and Gerburt-Geybovich, A. V. "Fundamentals of Soil Science and of Soil Mechanics." Moscow, 1956. 308 pp.
66. Kisselev, M. F. "On the Computation (No. 14): "Theory of Beams and Plates on a Compressible Base." The Moscow Civil Engineering Institute, 1956. 238 pp.
66. Kisselev, M. F. "On the Computation of Foundation Settlements on Thawing Soil Bases." Gosstroyizdat. Moscow, 1957. 50 pp.
67. Petrenko, G. M. "Mining Method of Installation of Deep Shaft Foundations." Gosstroyizdat, Ukrainian SSR. Kiev, 1957. 127 pp.
68. Popov, I. V. "Some Problems of Study of Clays for Engineering-Geological Purposes." Lvov State University, 1957. Pp. 291-293 (reprint).
69. Roukavtzov, A. M. and Perley, E. M. "Experiences in the Use of Vibratory Drivers in Industrial Construction." Leningrad, 1957. 78 pp.
70. Denissov, N. J. and Zhoukova, V. M. "Pore Pressure and the Resistance of Clay Soils to Shear." Moscow, 1957. 44 pp.
71. Denissov, N. J. "Engineering Geology and Hydrogeology." Gosstroyizdat. Moscow, 1957. 366 pp.
72. Istomina, V. S. "Seepage Stability of Soils." Moscow, 1957. 295 pp.
73. Tsytovich, N. A. et al. "Data on Laboratory Studies of Frozen Soils." A Symposium (No. 3) of the Permafrost Institute, Academy of Sciences USSR. Moscow, 1957. 323 pp.
74. Petoukhov, I. A. et al. A Symposium (No. XXXII): "Problems of Investigation of Rock Pressures and of Dis-

placement of Rock Deposits." All-Union Scientific-Research Markscheider Institute. Leningrad, 1958. 298 pp.

75. Ivanov, N. N., Ornatsky, N. V. and Babkov, V. F. "The IV International Congress on Soil Mechanics and Foundation Engineering." Avtotransizdat. Moscow, 1958. 180 pp.

76. Silin, K. S., Glotov, N. M., Gretsov, A. E., Kappinsky, V. I. and Prohorov, A. D. "Bridge Pier Foundations of Composite Reinforced Concrete Cylinders." State Transport Publishing House. Moscow, 1958. 200 pp.

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79. Ornatsky, N. V. "On the Deformability of Watersaturated Soils." Scientific Report of Higher Schools. 1958, No. 4. Pp. 182-187 (reprint).

80. Shneider, M. E. "Technical Terms in English, Russian, French, German, Swedish, Portuguese and Spanish used in Soil Mechanics and Foundation Engineering." Moscow, 1958. 140 pp.

81. Gorbounov, B. P. "Seepage and Possibility of Its Use for the Determination of the Specific Surface and Quantity of Bound Water in Watersaturated Soils." Izvestiya, Academy of Sciences USSR, Section of Technical Sciences, No. 9. 1958. Pp. 103-105 (reprint).

82. Maslov, N. N. "On the Problem of Conditions Causing the Appearance of Creep in Clay Soils of the Base of Retaining Structures." Scientific Reports of Higher Schools, 1958, No. 1. Pp. 104-118 (reprint).

83. Goldstein, M. N. "Generalization of Equations of Thermodynamics of Irreversible Processes." Academy of Sciences USSR, Journal of Physical Chemistry, Vol. XXXII, No. 9. 1958. Pp. 2219-2220 (reprint).

84. Ter-Stepanian, G. I. "Classification of Slide Cracks." Izvestiya, Academy of Sciences Armenian SSR. Erivan, 1958. Vol. XI, No. 5. Pp. 29-45 (reprint).

85. Rosa, S. A. and Kotov, A. I. "Experimental Studies of Soil Skeleton Creep." Journal of the Leningrad Mining Institute, Vol. XXXIV, No. 2. 1958. Pp. 203-213 (reprint).

86. Korenev, B. G. et al. Symposium: "Problems of Design of Slabs on an Elastic Base." Moscow, 1958. 121 pp.

87. "Instructions for the Determination of the Relative Compression of Frozen Soils when Thawed under Pressure." Scientific-Research Institute on Foundations. Moscow, 1958. 16 pp.

88. Efremov, M. G. "Vibromethod of Sinking Bore Holes for Geological Exploration." Scientific-Research Institute on Foundations. Moscow, 1958. 44 pp.

89. "Temporary Instructions for the Unwatering of Construction Cofferdams in Clayey Soils Using Wellpoints Installation and Direct Electric Current." Scientific-Research Institute on Foundations. Moscow, 1958. 16 pp.

90. "Lowering Ground Water Levels by Light Wellpoint Installations and Ejector - Wellpoints." (Instruction Manual) Scientific-Research Institute on Foundations. Moscow, 1958. 110 pp.

91. Egerev, K. E. "Design of Reinforced Concrete Frame Foundations and Frameworks for Uneven Settlement of Their Supports." Academy of Sciences USSR, Permafrost Institute. Moscow, 1958. 59 pp.

92. Doroshkevich, N. M. "On the Distribution of Stresses in Pile Foundations." Scientific Reports of Higher Schools. 1958, No. 4. Pp. 97-104 (reprint).

93. Vinokourov, F. P., Teterkin, A. E. and Peterman, M. A. "Peat in Construction Work." Minsk, 1959. 242 pp.

94. Litvinov, I. M. "Basic Requirements for the Design and Execution of Ther-

mic Soil Stabilization Works." Gosstroyizdat Ukrainian SSR. Kiev, 1959.

95. Goldstein, M. N. "Thermodynamics of Irreversible Processes and the Electro-osmotic Transfer of Ions." Academy of Sciences USSR, Colloidal Journal, Vol. XXI, No. 1. 1959.

96. Drannikov, A. M. "Engineering Geology." Gosstroyizdat Ukrainian SSR. Kiev, 1959. 221 pp.

97. Romanov, D. A. "Pile Foundations of Deep Penetration." Gosstroyizdat Ukrainian SSR. Kiev, 1959. 235 pp. (*Note:* Includes a Chapter on Camouflet Piles.)

98. "Recommendations on the Organization and Performance of Observations Concerning Changes in the Water-Temperature Regime of Permanently Frozen Soils for Purposes of Foundation Construction." Scientific - Research Institute on Foundations. Moscow, 1959. 28 pp.

99. Barkan, D. D. "Vibromethods in Construction Work." Gosstroyizdat. Moscow, 1959. 316 pp.

100. Tourovskaya, A. Y. "Study of the Strength of Primary Kaolins." Institute of Railway Transport Engineers. Dniepropetrovsk, 1959. 20 pp.

101. Gorbounov-Possadov, M. I. "Tables for the Design of Thin Plates on an Elastic Base." Gosstroyizdat. Moscow, 1959. 97 pp.

102. Tsytovich, N. A., Chairman of Editorial Board. "Fundamentals of Geocryology. Part I, General Geocryology." Permafrost Institute, Academy of Sciences USSR. Moscow, 1959. 460 pp.

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