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MISSING

The Canadian Engineer

A weekly paper for Canadian civil engineers and contractors

Effects of Grading of Sands and Consistency of Mix Upon the Strength of Concrete

Increment of Strength Developed Between Ages of Ninety Days and One Year Not Constant and Cannot be Predetermined—Excess Water Reduces Resistance to Abrasion and Shock—Paper Read Before American Society For Testing Materials

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THIS paper is primarily intended as a supplement to the writer's paper presented under the same title at the last annual meeting of the American Society for Testing of Materials.† The curves showing the compressive strengths of the test cylinders for both the "Tests for Grading of Sands" and the "Tests for Consistency of Mix" have been revised to include the strength of cylinders tested at the age of one year. The information relating to the effect of consistency of mix is augmented by the addition of the results of a test made to show the decrease of the resistance to abrasion and shock resulting from the inclusion of a water content in the mix in excess of that required "to produce a concrete, the mortar component of which is of a saturated, sticky, semi-plastic consistency."

In 1917 a series of tests was made by the City of Toronto, Department of Works, to secure information relating to the strengths to be obtained from concretes in which gravel was used as aggregate. The description of materials, methods, etc., together with the results secured from these tests, are fully described in an appendix to this paper. The sand and gravel used came from the same pit as did the sand used in the 1916 broken-stone concrete tests. All tests were made under the direct supervision of the writer.

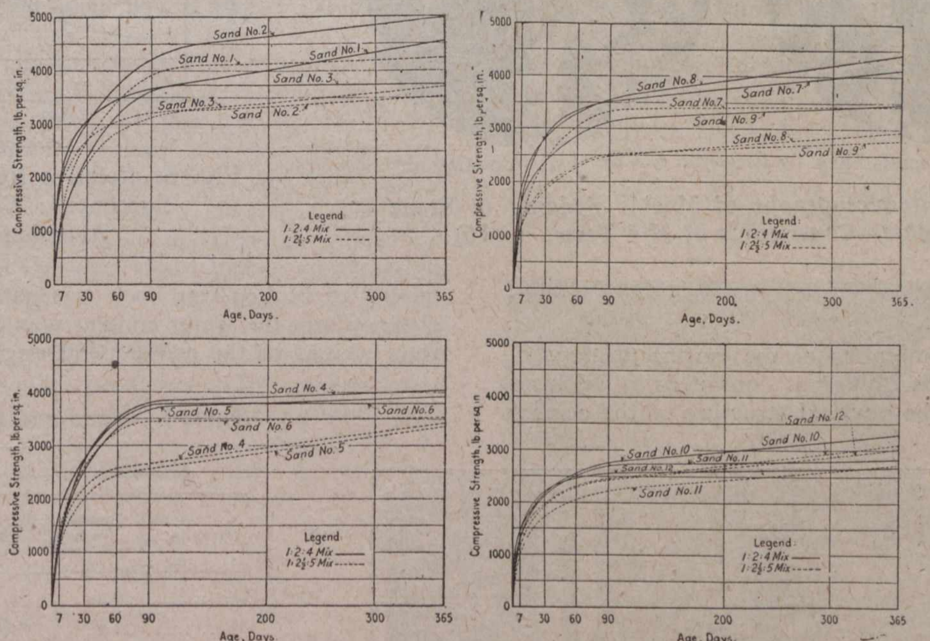


Fig. 1—Compressive Strengths of Test Cylinders: Grading of Sands Test

In plotting the curves showing the compressive strengths of test specimens for ages of 90 days and under, the strengths obtained from the 90-day tests were in all cases assumed to be unchangeable and in a few cases slight variations from plotted strengths at 7, 30 and 60-day ages were made to secure uniform, even curves. In revising these strength curves to include the 1-year test strengths it has been found advisable, in a few cases and for the same reason, to vary from the plotted strengths at 90 days.

Tests for Grading of Sands. Fig. 1 shows the compressive strengths obtained from the tests of the cylinders in which the twelve specially graded sands were used.

Tests for Consistency of Mix.—Fig. 2 shows the compressive strengths obtained from the tests of the cylinders in which the consistency of mix was varied from a sticky, semi-plastic to a very wet condition. In relation to the total weight of the cement and aggregates the quantity of water used in the first consistency mix was 6.17 per cent. or the 1 : 2 : 4 mix and 5.99 per cent. for the 1 : 2½ : 5 mix. This quantity was increased by 10, 20, 35 and 50 per cent., respectively, for each succeeding mix.

In the previous paper mention was made of the condition that "the inclusion of a greater quantity of water than is required for the development of its natural functions is decidedly detrimental to the strength and reliability of the final concrete," and photographs were

†Capt. Edwards' previous paper was printed in full in *The Canadian Engineer*, issues of August 16th, August 23rd, August 30th and September 6th, 1917.

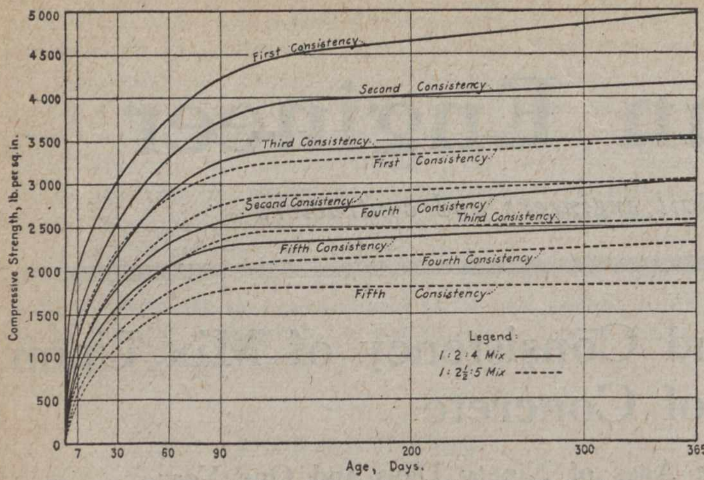


Fig. 2—Compressive Strengths of Test Cylinders: Consistency of Mix Tests

reproduced showing accumulations of water cavities underlying the particles of stone aggregate and the reinforcing steel tending to reduce the bearing capacity in the former case and the bond between the steel and the concrete in the latter case.

A microscopic examination of the mortar portions of the concretes of the five different consistencies discloses a marked change in the physical structure of the cement matrix. Doubtless the decrease in strength found in the tests of concrete cylinders and of reinforced concrete beams is in part due to this change in the matrix.

(*a*) First Consistency (*b*) Fifth Consistency

Fig. 3—Photomicrographs of Mortar Bedding of Stone Aggregate—Consistency of Mix Tests. (×75)

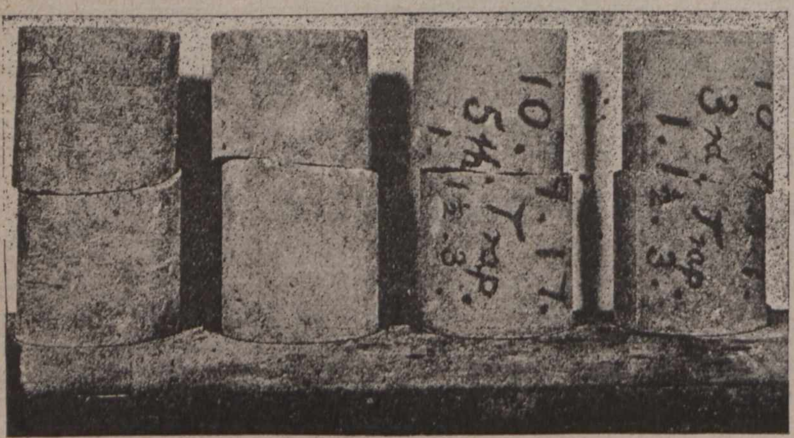


Fig. 4—Test Specimens: Consistency-Abrasion Test

Fig. 3 shows photomicrographs (×75) of the mortar bedding of stones in first and fifth consistency concretes. Under the microscope the cement matrix of the first consistency concrete appears gray in color and adheres closely to the surfaces of the aggregates in amorphous masses containing myriads of minute voids, but on the whole giving the impression of possessing strength, hardness and rigidity. In comparison, the cement matrix of fifth consistency concrete presents a marked difference, both in color and in physical structure. Its color is more nearly a milky white and its structure is generally loose and flaky, giving ample evidence of a lack of strength resulting from absence of cohesion between the cement particles and adhesion between these particles and the surfaces of the aggregates.

Consistency-Abrasion Tests.—This test was made with the object of obtaining information relating to the influence of the consistency of mix upon concrete in relation to its resistance to abrasion and shock. The mix used was a 1:1½:3. Two sets of test cylinders were made. In the first set the stone aggregate was broken limestone of good quality and in the second set a good quality of broken trap rock was used. In the first consistency mix the water content was 6.25 per cent. of the total weight of the cement and aggregates. The water content was increased by 10, 20, 35 and 50 per cent. for the four succeeding consistencies. The test specimens were cast in 6-in. cylinder forms by the insertion of a

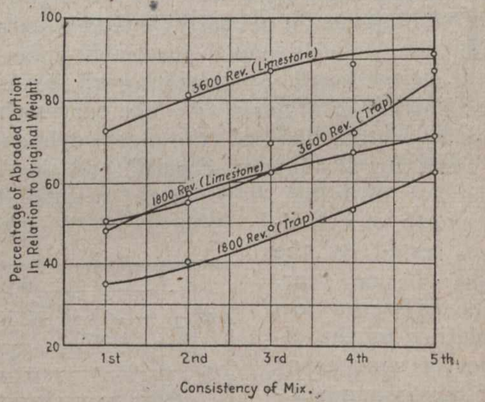


Fig. 5—Relative Reductions of Test Specimens: Consistency-Abrasion Test

Table I.—Sand and Gravel Gradings

Sand Grading.		Gravel Grading.	
Sieve No.	Percentage Retained on Sieve.	Screen Size, in.	Percentage Retained on Screen.
4	0.0	2	0.0
8	14.0	1½	15.0
10	4.0	1	26.0
20	14.0	¾	20.5
30	9.0	½	14.0
40	11.0	¼	24.5
50	14.0		
80	20.0		
100	6.0		
200	6.0 2.0 (passing)		

metal diaphragm at about the centre of the length. Fig. 4 shows the specimens so produced. Ten half cylinders were made for each consistency.

The specimens were tested at the age of three months by the "rattler" abrasion test commonly used for testing paving brick. Fig. 5 shows the percentages of reduction in weight resulting from 1,800 revolutions and from 3,600 revolutions of the machine. In this test several of the "wet" consistency specimens were entirely pulverized.

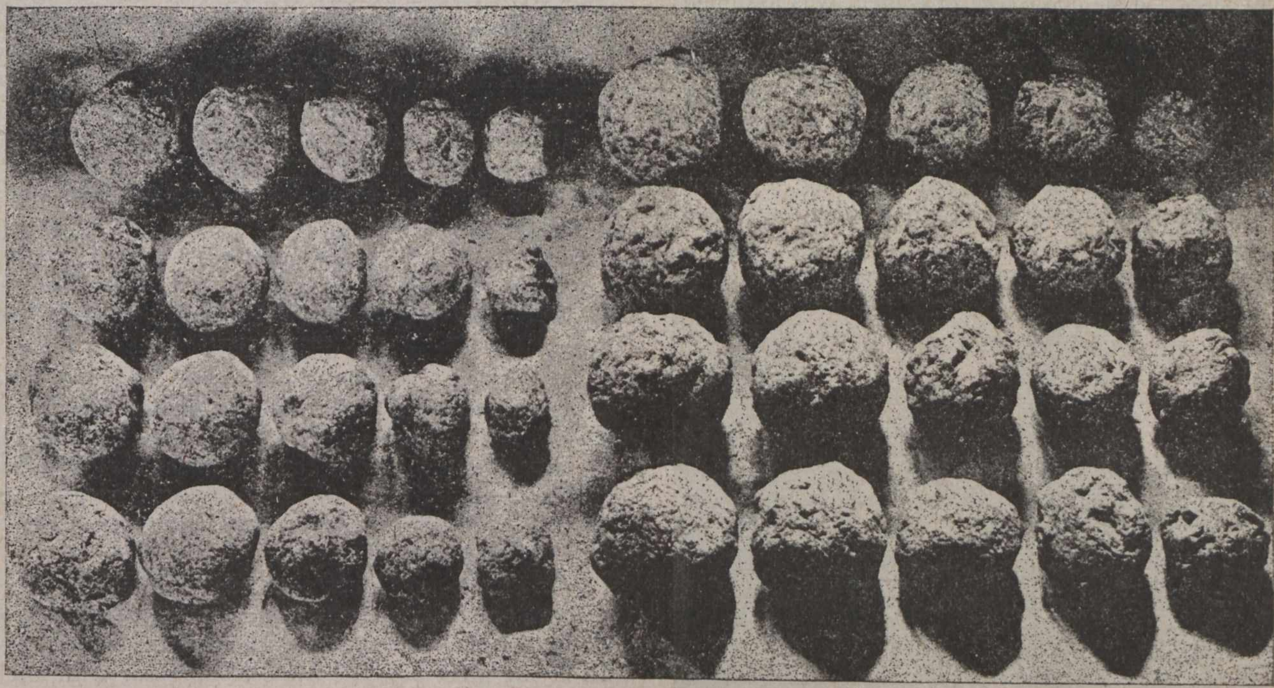


Fig. 6—"Rattled" Test Specimens: Consistency-Abrasion Tests. *(Limestone Aggregate Specimens at Left, Trap Aggregate Specimens at Right. Consistencies Arranged in Order from Left to Right)*

Fig. 6 shows typical specimens for each consistency after final removal from the testing machine. The trap concrete specimens are shown at the right; the limestone concrete specimens at the left. Consistencies are arranged in order from left to right.

Conclusions

The one-year tests have produced no information from which the writer finds reason for modifying the conclusions contained in his 1917 paper. However, this further conclusion seems pertinent.

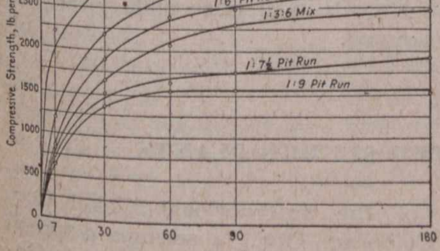


Fig. 7—Compressive Strengths of Gravel Concretes of Different Mixes

The increment of strength developed between the age of 90 days and 1 year is not constant and cannot be predetermined. In so far as "wet" concretes are concerned this increment is almost negligible. The results obtained in the consistency-abrasion test indicate that the excess water in an over-saturated concrete reduces its resistance to abrasion and shock.

Appendix

Strength Tests of Gravel Concrete.—In the concrete tests herein described the aggregates were secured from the Maple Sand and Gravel Co., Maple, Ont. The sand used in 1916 in connection with the "Grading of Sands" and "Consistency of Mix" tests described in the writer's paper last year was secured from the same company.

Throughout this series of tests the sizes of test specimens and the methods of proportioning, mixing, placing, storing, testing, etc., were the same as those used in the above-mentioned tests. In all cases only a sufficient quantity of water was used to produce a saturated, semi-plastic mortar adhering freely to the gravel particles. These tests are, therefore, directly comparable with the tests previously made in so far as similar aggregates of limestone origin were used in both cases.

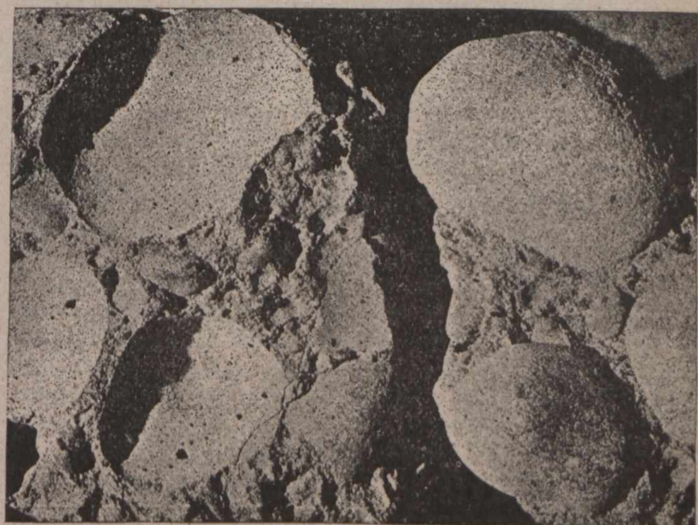


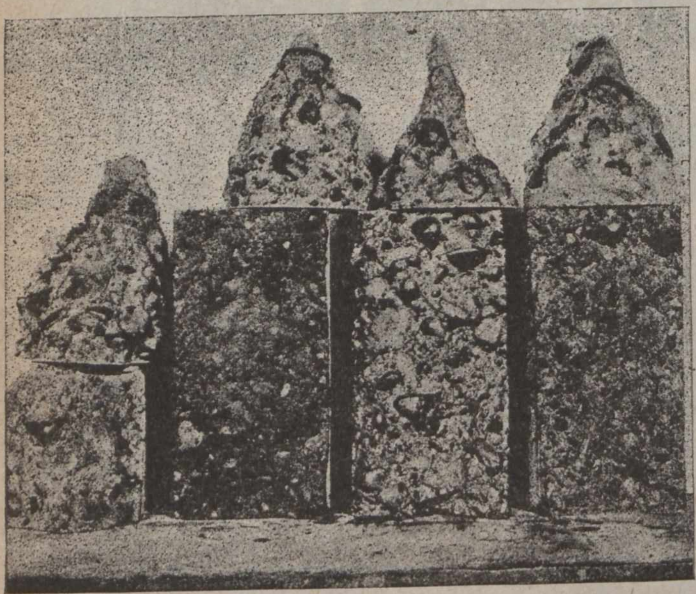
Fig. 8—Surface Contact Between Under Side of Gravel Particles and Mortar Bed, 1:2:4 Mix

Materials Used in Tests

Cement.—As compared with the cement used in the 1916 tests, the "Canada" brand cement used in these tests developed in the 7 and 28-day tests a strength 10.8 per cent. less than that used in 1916, the same standard tests being used. One cubic foot of cement was assumed to weigh 100 lbs.

Sand and Gravel.—The pit run gravel secured for the tests contained by weight 65.2 per cent. of sand passing a No. 4 sieve and 34.8 per cent. of gravel. The gradings of these portions are shown in Table I.

In its original state this material contained 7.59 per cent. of moisture, of which the sand portion contained



(*d*) (*c*) (*b*) (*a*)

Fig. 9—Broken Concrete Test Cylinder of Various Mixes

(*a*) Contains Broken Stone (Limestone) Aggregate, 1:2:4 Mix.
(*b*) Gravel Concrete, 1:2:4 Mix.
(*c*) Gravel Concrete, Pit Run, 1:6 Mix.
(*d*, upper) Gravel Concrete, 1:2½:5 Mix.
(*d*, lower) Gravel Concrete, Pit Run, 1:9 Mix.

5.86 per cent. This moisture was not dried out but was taken into account in proportioning the sand and the water contents of the mix.

The unit weights assumed for proportioning the mix were as follows:

One cubic foot of sand assumed at 100 lbs.
One cubic foot of pit run gravel assumed at 110 lbs.
One cubic foot of screened gravel assumed at 110 lbs.

The screened gravel and the sand used in the regularly proportioned mixes were obtained by passing the pit run material over a screen having four meshes per linear inch.

Results of Tests

The tests of a 1:3 "normal" mortar, in which the aggregate was the sand used in these tests, showed at 28 days' age a tensile strength of 412 lbs. and a compressive strength of 3,312 lbs. per square inch. Briquettes and 2-in. cubes were used for these tests.

Fig. 7 shows the compressive strengths obtained from the tests of the 6-in. cylinders prepared from each mix. Five specimens were tested at each age of 7, 30, 60, 90 and 180 days.

Fig. 8 shows the mortar bedding of gravel particles in a 1:2:4 mix. The gravel particles removed from the beddings are shown at the right side of the photograph. This photograph is typical of the beddings found in specimens of all mixes. However, the porosity and the color of the mortar portion varied with the quantity of cement in the mix.

Fig. 9 shows broken cylinders of various mixes. It will be noted that the portions of gravel concrete cylinders contain very few broken particles, while the cylinder at the extreme right containing broken stone aggregate shows a considerable number of fractured particles. This failure of the mortar portion of the concrete, leaving the gravel particles intact, was typical of all mixes. The excess of mortar in the pit run specimens is clearly shown in the photograph.

LABOR-SAVING MACHINERY IN WATERWORKS SERVICE

In a paper read before the Minnesota Section of the American Waterworks Association, J. A. Jensen, superintendent of the Minneapolis Waterworks, pointed out the value of labor-saving devices in connection with waterworks operation, from which the following extracts are taken:—

On work in limestone ledge, trenching for pipe-laying, an air compressor and operator with two plug-drills and two men replaced twenty men with hand-drills. In laying steel pipe the same compressor with ten boiler-makers equipped with the necessary riveting and caulking hammers, reaming tools and other equipment carried on the work of a whole company of similar skilled labor with hand tools.

A single first-class power-operated trench pump has been found to do as much work as three hand-operated pumps manned by a dozen laborers.

In 1913 the city of Minneapolis purchased a large trenching machine for use on several miles of trunk pipe lines. This machine put out 39,200 cu. yds. of earth at a unit cost of 9.3 cents per cubic yard on about three miles of trench. On another piece of work a steam shovel on back-fill handled 23,730 cu. yds. on 9,250 ft. of trench at a unit cost of 4.8 cents per cubic yard.

In service work at Minneapolis with miscellaneous jobs scattered over forty or more square miles of territory, Ford runabout trucks are employed, each rig and two men, generally speaking, replacing three single-horse rigs and six men. This is explained by the fact that the principal part of the performance in street service is mileage, and the work itself is secondary, so far as time element is concerned.

ACTIVATED SLUDGE PLANTS

At the new plant of the International Nickel Co., Limited, Port Colborne, Ont., a sewage disposal plant using the activated sludge process will be installed, the company having come to terms with Messrs. Jones & Attwood, Limited, of Stourbridge, England, the owners of the activated sludge patents.

The authorities of the City of Manchester, England, have secured the sanction of the War Cabinet to carry out immediately, at their main outfall at Davyhulme, an activated sludge scheme for treating a million gallons of sewage per day.

RECOMMENDED PRACTICE FOR CONCRETE BRIDGE CONSTRUCTION

THE following general specifications have been prepared by the engineers of the Portland Cement Association. They are based on the recommendations of the Committee on Bridges of the American Concrete Institute and on parts of Bulletin No. 10 of the Illinois State Highway Department, "A Manual for County Superintendents of Highways, Resident Engineers, and Inspectors."

Materials

CEMENT.—The cement should meet the requirements of the current standard specifications for portland cement of the American Society for Testing Materials. Adequate provision should be made for storage of cement on the work and for protection against dampness in storage.

AGGREGATES.—Before aggregates are delivered on the job, contractors should submit to the engineer a 50-lb. sample of each of the aggregates proposed for use. These samples should be selected so as to represent fairly the average quality and grading of the aggregates in question. The samples submitted should be tested and, if found to pass the requirements of the specifications, similar material should be considered as acceptable for the work. Aggregate containing frost or lumps of frozen materials must not be used. Fine and coarse aggregate should not be piled so near each other that the edges of the piles overlap. Aggregate should perferably be stored on a platform so that the material in the bottom of the pile will not become mixed with dirt during wet weather, thus lowering the strength of the concrete.

FINE AGGREGATE.—Fine aggregate should consist of natural sand or screenings from hard, tough, crushed rock or pebbles, graded from fine to coarse, with the coarse particles predominating. Fine aggregate, when dry, should pass a screen having four meshes per linear inch. Not more than 25 per cent. should pass a sieve having 50 meshes per linear inch, and not more than 6 per cent. should pass a sieve having 100 meshes per linear inch. Fine aggregate should not contain injurious vegetable or other organic matter, as indicated by the colorimetric test,* nor more than 7 per cent. by volume of clay or silt as determined by washing the sand thoroughly in 100 per cent. excess of water and allowing the silt to settle for one hour. Field tests should be made by the engineer on the fine aggregate as delivered at different times during the progress of the work. If the above requirements regarding organic matter and silt are not fulfilled, the material represented by the samples should be rejected.

Fine aggregate should be of such quality that mortar composed of one part of portland cement and three parts fine aggregate, by weight, when made into briquettes or cylinders, shows a tensile or compressive strength at 7 and 28 days at least equal to the strength of briquettes or cylinders composed of one part of the same cement and three parts of standard Ottawa sand by weight. The proportion of water used in making the test pieces of cement and fine aggregate should be such as to produce a mortar of the same consistency as that of the Ottawa sand test-pieces or normal consistency. In other respects briquettes or cylinders should be made in accordance with the methods of testing cement recommended by the American Society for Testing Materials.

*The colorimetric test is described fully in Bulletin No. 1, entitled "Colorimetric Test for Organic Impurities in Sands," issued by the Structural Materials Research Laboratory, Lewis Institute, Chicago. This bulletin may be obtained from the Portland Cement Association.

COARSE AGGREGATE.—Coarse aggregate should consist of clean, durable, crushed rock or pebbles graded in size, free from vegetable or other organic matter and should be practically free from soft, flat, or elongated particles. The coarse aggregate should be well graded from 1½ ins. down (1 in. for reinforced concrete), not more than 5 per cent. passing a screen having four meshes per linear inch.

RUBBLE STONE.—For mass work, "one-man" stones may be embedded in the concrete provided that they are not placed nearer than 6 ins. to the outer surface of the concrete and not closer to each other than 6 ins. The quantity of such stone should not exceed 30 per cent. of the cubical content of the wall or foundation.

MIXED AGGREGATE.—Crusher-run stone, bank-run gravel or mixtures of fine and coarse aggregate prepared before delivery on the work should not be used, because the ratio of fine to coarse material varies so widely as to lead to mixtures of greatly varying proportions.

WATER.—Water should be clean and free from oil, acid, alkali, or vegetable matter.

REINFORCEMENT.—Reinforcing metal should meet the requirements of the current standard specifications for steel reinforcement of the American Society for Testing Materials. It should be free from excessive rust, scale, paint, or coatings of any character that would tend to reduce or destroy the bond. Either plain or deformed steel bars should be used unless otherwise specified by the engineer. When deformed bars are used they should have a net cross-sectional area equivalent to that of the plain bars shown on the plans. All reinforcing metal should be so stored and handled at the work as to ensure its being placed in the structure in a clean condition.

JOINT-FILLER.—The expansion-joint filler should be either a suitable elastic waterproof compound, which will not become soft and run in hot weather, nor become hard and chip in cold weather; or else prepared strips of fibre matrix and bitumen as approved by the engineer.

Construction

FORMS.—Forms should be substantial, unyielding, and so constructed that the concrete conforms to the designed dimensions and should also be tight to prevent leakage of mortar. The forms for exposed surfaces should be sound lumber, planed to a uniform thickness, either tongued and grooved, or with edges beveled to make tight joints after erection. Special attention should be given shores for beams, girder, and slab work, wall-ties, bracing for walls, and centring for arch-form supports, in order to ensure the completion of the work without sagging and deformation. The supports for bridge floors, beams, girders, and arches should remain in place until time for removal, as hereinafter recommended. Forms should be filleted at all sharp corners. For all projections such as girders and copings, the forms should be given a bevel or draw sufficient to ensure easy removal. Forms should be painted with boiled linseed oil or other equally good preparation to prevent the concrete from adhering to the same. A coat of ordinary whitewash is effective. Crude oil or any material that will stick to or discolor the concrete, such as tar-paper, should not be used. Care should be taken to secure perfect alignment of rail and girder forms. Local kinks should be eliminated before the concrete is placed.

PLACING REINFORCEMENT. — Reinforcement should be placed in the position shown on the plans and should be wired at intersections so that it will not become disarranged during the depositing of the concrete. Where bars are spliced, a length of lap sufficient to develop the full strength of bar should be used. If slips of an approved design or other method of connecting rigidly the bars of a reinforcing member be used, the length of lap may be decreased an amount depending upon the nature of the connections. Splices at points of maximum stress should be avoided. Steel should not be laid directly on forms and raised during placing of concrete by prying it up and working concrete under it. The steel should be laid on small stones or blocks of concrete, which may remain when the concrete is placed. In general, steel should be placed securely before concrete is deposited.

Measuring and Mixing

MEASURING.—The method of measuring the materials for the concrete or mortar, including water, should be one that will ensure separate and uniform proportions of each of the materials at all times. Uniformity of proportions for the aggregates can best be maintained by the use of bottomless measuring boxes or wheelbarrows whose capacities have been determined. A sack of portland cement (94 lbs. net) should be considered as one cubic foot.

MACHINE MIXING.—Concrete should be mixed by machine except when the engineer permits otherwise under special conditions. A batch-mixer of an approved type should be used. The ingredients of the concrete or mortar should be mixed to the specified consistency. The mixing should continue at least one minute after all materials are in the drum before any part of the batch is discharged from the drum. The drum should be emptied completely before receiving materials for the succeeding batch. The volume of the mixed material used per batch should not exceed the manufacturer's rated capacity of the drum in cubic feet of mixed material. The mixer should be equipped with water storage and a measuring device which can be locked, also with a suitable charging-hopper.

HAND MIXING.—When it is necessary to mix by hand, the materials should be mixed dry on a water-tight platform until the mixture is of uniform color, the required amount of water added, and the mixing continued until the mass is of uniform consistency and is homogeneous. The concrete should be placed in the work within 30 minutes after the water is added to the dry materials.

RE-TEMPERING.—Re-tempering of mortar or concrete that has hardened partly, that is, re-mixing with or without additional materials or water, should not be permitted.

Gravity Abutments, Wing-Walls, and Piers

PROPORTIONS.—Concrete for abutments and wing-walls should be mixed in the proportions of one sack of portland cement to not more than two and a half cubic feet of fine aggregate and five cubic feet of coarse aggregate.

CONSISTENCE.—Sufficient water should be added to the dry materials to produce a concrete of a consistence such that it will flow slowly into the forms, but can be handled to place without a separation of the coarse aggregate from the mortar. This consistence is best described as "quaky."

PLACING.—Concrete should be placed in a manner that will permit the most thorough compacting and should be worked and spaded into all recesses of the forms. Concrete should be deposited in its final position as soon as possible after mixing and before it has begun to harden. When the work is interrupted so that the last layer of the concrete has hardened before the next can be applied, the upper surface should be left level, roughened, and cleaned, and should then be covered until the work is resumed, at which time the surface should be cleansed of all foreign material and laitance, then drenched with a mortar consisting of one sack of portland cement and two and a half cubic feet of fine aggregate. The concrete should be placed in continuous horizontal layers. Vertical joints should be avoided whenever possible. Concrete should never be placed in running water. In still water a tremie or other apparatus that will ensure the placing of the concrete without separation of the cement and aggregate and the formation of an injurious amount of laitance should be used. Concrete should never be placed in water when it is possible to avoid so doing.

Arches, Bridge Floors, Beams and Girders, Reinforced Cantilever-Type Substructure

PROPORTIONS.—Concrete should be mixed in the proportions of one sack of portland cement to not more than two cubic feet of fine aggregate and four cubic feet of coarse aggregate.

CONSISTENCE.—Same as under "Gravity Abutments."

PLACING.—Concrete should be placed in a manner to ensure a smooth surface next to the forms and should be spaded thoroughly and worked around the metal reinforcement and into the recesses of the forms. If possible, concrete for an arch-ring should be deposited in a single run. Large arch-spans, where it is impossible to cast the arch-ring continuously, should be divided into transverse sections care being taken that the bulkheads dividing the sections are placed radially. Concreting should then start at the crown or springing lines and should be done simultaneously or alternately in such sections as will nearly balance the load and thrust and thus prevent distortion of the form work and supports. If possible, the entire floor should be placed in one day. If not, the dividing line should be made in a vertical plane, perpendicular to the main reinforcing rods (in slab bridges at right angles to the axis or roadway and in floors of girder bridges parallel to that axis). Concrete should be deposited to the full thickness of the floor and should be brought to the surface of the established grade of the bridge floor. Workmen should not be allowed to walk on the freshly laid concrete.

Surface Finish

The top surfaces of all walls, abutments, girders, slabs, hand-rails, and copings should be tamped carefully and thoroughly. The top surface of abutments, hand-rails, and copings should be given a slightly arched contour, obtained by striking off the concrete in the forms with a strike-board cut to give this surface the desired contour. Concrete in floor slabs that are to serve as the finished road surface should be compacted by rolling with a finishing roller similar to the Macon concrete paving roller.* Immediately after the forms have been removed, projections and irregularities on exposed surfaces should be removed and cavities neatly filled with mortar. Enough 1 : 2 cement-and-sand mortar should be used to fill the cavities, but no plastering should be permitted. All ex-

*A circular describing this roller and the method of using it can be obtained from Portland Cement Association.

posed surfaces should be finished to give a smooth and neat appearance, and when specified should be scrubbed or otherwise finished in an approved manner as directed by the engineer. The bridge floor, after being brought to the established grade and struck off with a template or strike-board, should be finished with a wooden hand-float in such a manner as to compact it thoroughly and produce a surface smooth and free from depressions.

PROTECTION.—Surfaces should be protected from sun and wind and the concrete sprinkled in dry weather so that the whole surface is kept wet for a period of at least one week. Floors should preferably be covered with damp earth, which should be kept wet by frequent sprinkling for at least ten days or until thorough hardening of the concrete is ensured.

TEMPERATURE BELOW 35° F.—If at any time during the progress of the work the temperature is below, or in the opinion of the engineer will within 24 hours fall to, 35° F., the aggregates and mixing water should be heated and adequate precautions taken to protect the work from freezing for at least seven days.

HOT WEATHER.—Freshly placed concrete exposed to hot weather and strong sunlight should be protected by watering the surfaces frequently and covering for seven days after being placed.

REMOVAL OF FORMS.—No forms should be removed within 48 hours after concrete is placed. Supports for floors, slabs, and bridge arches should remain in place at least 21 days. When freezing weather occurs the supports should remain in place for an additional time equal to the time the structure has been exposed to freezing temperatures. No extraneous loading should be placed on the concrete before removal of the forms. The bridge should not be opened to traffic until the engineer directs.

FILLING AGAINST WALLS AND ARCHES.—Earth filling against abutment, wing, and spandrel walls should be placed in horizontal layers, avoiding wedge-shaped sections against the walls. Arch-fillings should be deposited in such a manner as to load the arch symmetrically. The filling back of abutments and between spandrel walls should be drained thoroughly by means of drainage lines and scuppers through wing-walls, arch-rings, and piers.

DRAINAGE OF FLOORS.—The floors of all slab and girder bridges should be provided with suitable drain-scuppers at frequent intervals.

COFFER-DAMS.—Coffer-dams should be constructed as nearly water-tight as practicable. No pumping should be permitted inside of foundation forms while concrete is being placed. Should it become necessary to prevent flooding, a seal of concrete may be deposited in the bottom of the coffer-dam and allowed to set. Methods of depositing concrete must be such as to ensure that there shall be no separation of materials. If closed chutes or troughs are used, these should never be set at so flat an angle that it will be necessary to use a concrete of wetter consistence than that required for best results. Arch foundations on piling should be enclosed in permanent continuous sheet piling, having the tops sawed off one foot below low water.

Foundations

EXCAVATION.—All footing excavations should be carried down to firm material except where the sub-foundation material is known to be soft enough to require the use of piles or other special treatment.

LOAD ON FOOTINGS.—The footings shown on plans prepared by the Illinois Highway Commission are spread to such an extent as to make the average bearing on the sub-foundation material not more than 1½ tons per square foot of footing area. This load is safe for ordinary material such as firm clay, coarse sand, gravel, or even moderately compacted made ground. It is not an overload for confined sand, provided there is no danger of the stream-bed scouring so as to release the sand or undermine the footings. Saturated sand, however, flows readily if unconfined. Sandy stream-beds are frequently subject to local scour which may endanger the foundations. Local scour 8 to 10 ft. below the stream-bed, because of some obstruction such as drift-wood or a sharp change in the course of the current, is not uncommon. Scour 20 ft. deep has been known. Abutment and pier foundations in sandy soils should therefore be designed with this danger in mind. Driving piles in sand with the aid of a water jet is not difficult and affords a fairly economical means of providing for such conditions.

SUBSOIL TESTS.—When the excavation is carried nearly to the depth called for on the plans, soundings should be made by driving a rod in the bottom of the excavation, or, preferably, test-pits should be dug to determine the character of the underlying material.

EXTRA FOOTINGS.—It is not usually economical to carry the footings to a depth greater than 3 ft. more than that shown on the plans, as the contract price for extra concrete in footings is usually high. If firm material cannot be secured at a depth 3 ft. or less below that shown on the plans, it is usually more economical to drive piles.

BRACING PITS.—In making excavations under ordinary conditions, the sides of the excavation if vertical, and more than five or six feet high, are apt to cave unless thoroughly braced. It is usually cheaper and undoubtedly safer for the contractor either to excavate in the first place to a 1 to 1 or flatter slope or else to arrange to brace the trench thoroughly.

Piles

LOAD ON PILES.—When piles are used, it is good practice to drive a sufficient number to carry the entire load, including live load and weight of bridge complete, so that the abutment will be stable even should scour remove part of the soil from underneath the footing.

TEST-PILE.—Should circumstances permit, a test-pile may be driven before the piles are ordered. The test-pile should ordinarily be not less than 20 ft. long. During driving, the necessary observations should be made for the determination of its supporting capacity by the Engineering News' formula. Using the results so found, the number and length of piles may be determined.

Concrete Footings

INSPECTION OF COMPLETED EXCAVATION. An inspector should be on the work when the placing of concrete in the footings is started in order to be sure that the sub-foundation material is satisfactory and that the footing concrete is properly placed. The bottom of the excavation should be clean and as free from soft material as possible.

TEMPERING FOOTING CONCRETE.—Dry concrete should never be placed in footings, even should the excavation contain considerable water. If concrete is placed dry, it is likely to become wet only on the surface, leaving most of the concrete dry.

PLACING CONCRETE IN FOOTINGS.—If there is trouble from an excessive amount of water draining into the pit, care should be taken to prevent the cement from being washed out of the concrete and carried off with the water removed by the pump. If there are no steel rods to interfere, the footings should be carried forward to the full depth from one end of the trench toward the pump.

USE OF TILE.—It may be necessary to lay a line of tile around the edge of the excavation to take the water to the end where the suction of the pump is placed. Concrete may then be placed, beginning at the opposite end from the pump suction, finally stopping the pump and filling the pocket or sump at the point of suction. This layer should be allowed to harden for 24 hours or more before pumping out the excavation again.

Reinforced Footings

PLACING.—In reinforced concrete footings a layer of concrete should usually be deposited as far as the elevation occupied by the lower bank of horizontal rods. These rods should then be placed with the vertical bars hooked around them and supported temporarily, and the next layer of concrete placed immediately before the lower layer has had time to set. The top rows of bars should then be placed and the footing concrete completed before any of the concrete hardens, working from one end of the abutment toward the other. If there is much trouble from water in the pit, it may be necessary to support all of the footing steel rigidly in place before any concrete is placed. This is good practice in any case.

SUPPORTING STEEL.—Projecting ends of reinforcing bars should be supported rigidly so as to avoid danger of the bond being loosened. This applies particularly to the vertical bars in abutments, wings, and to stirrups in the girders. Thrusting bars to place in concrete that has attained initial set should not be tolerated.

PLACING STEEL.—Care is necessary in placing in exact position footing bars and the vertical bars that project from the footing as the section of greatest stress in a cantilever type of retaining wall is at the top surface of the footing and the part of the vertical bars below this section must therefore provide sufficient bond to develop the full strength of the bar.

Falsework

MUDSILLS.—Falsework for supporting the superstructure of concrete bridges should be designed, if on mudsills so that the sills have a sufficient bearing area to carry the weight of the superstructure without appreciable settlement. Not more than two tons per square foot of bearing area should be permitted for such sills on good material. A higher allowance may be made on firm hard material or gravel. On soft material the bearing pressure should be reduced by increasing the dimensions of the sills, or preferably piles should be driven to support the falsework, or possibly concrete footings placed.

CONCRETE FALSEWORK SILLS.—Contractors frequently find concrete footings for falsework to be advantageous. These may be constructed of lean concrete, but they should have ample area to carry the weight of the falsework and superstructure. Plenty of time should be allowed for the concrete to harden.

STRESSES IN FALSEWORK.—In order that undue deflection or crushing of the caps, stringers, and sheeting for the superstructure concrete may be avoided, the following stresses should not be exceeded for yellow pine or oak timber:

Fibre stress in cross-bending, 2,000 lbs. per square inch.

Crushing across the grain, yellow pine, 600 lbs. per square inch.

Crushing endwise of the grain for short blocks or stiff columns, 4,000 lbs. per square inch.

Long posts must be braced in both directions. The unsupported length in any direction should not exceed 15 diameters of the stick. All of the above stresses should be reduced one-half for green timber and for sapwood.

RULES FOR OPERATING IMHOFF TANKS

THE Committee on Sanitation of the Minnesota Surveyors and Engineers Society recently submitted a report covering rules for operating Imhoff tanks which are as follow:—

Any material, other than sand and grit, which may collect in the inlet chamber, should be forced through the opening into the settling chamber. Sand and grit should be removed and deposited upon the surface of the ground. The inlet chamber should be kept clean at all times. Any material which rises to the surface of the sewage in the settling chamber which does not sink readily when broken up, should be removed to the vents. Any sludge which deposits on the walls of the settling chamber should be forced through the slots into the sludge chamber. The walls of this chamber can be kept clean by daily scraping and forcing any adhering deposits through the slots. This can be accomplished by using the device shown on the appended blueprint.

Any scum which collects on the surface of the sewage in the vents should be thoroughly stirred up daily, so as to liberate the entrained gases and facilitate the settling of the solid material. If the scum in the vents should reach a thickness of over 6 ins. a portion of it should be removed. The surface of the sludge in the sludge chamber should never be allowed to reach a point higher than 2 ft. below the slots, equivalent to a point about 9 ft. below the surface of the sewage. The level of the sludge in the sludge chamber should be lowered about 2 ft. each time the sludge is removed, except in the late fall, when about 4 ft. should be removed, to allow for the winter's accumulation. The operator should be provided with a sampling outfit by means of which bottle samples of sewage in the sludge chamber can be collected at various depths and the exact location of the surface of the sludge determined.

The surface of the sludge should be determined at least once every two weeks.

Ice which may collect on the surface of the sewage in the settling and vent chambers should be removed immediately.

When the sludge on the drying bed reaches a depth of about 1 ft., it should be removed. Since a portion of the sand will adhere to the sludge removed from the bed, it will be necessary to replace the top layer of sand on the bed from time to time.

Particulars regarding the wet lignites of Valdarno, which are used in Italy for heating a Martin furnace at the Valdarno Foundry Works, are given in "L'Ingegneria Italiana. The lignite is distilled in a suitable furnace. The results obtained with the first plant were very good, in spite of the bad quality of the fuel, and its high percentage of water. With a 30-ton furnace three melts of steel are produced daily.

CONCRETE PAVEMENTS*

By Murray A. Stewart
Assistant Engineer in Charge of Roadway Section,
Dept. of Works, Toronto.

IT is not the intention in this article to institute any comparison between the usefulness of concrete and other materials for pavement purposes, as it is merely a recital of what might be called experimental work undertaken in order to ascertain the respective merits of different classes of concrete, and different methods of construction adopted in connection therewith.

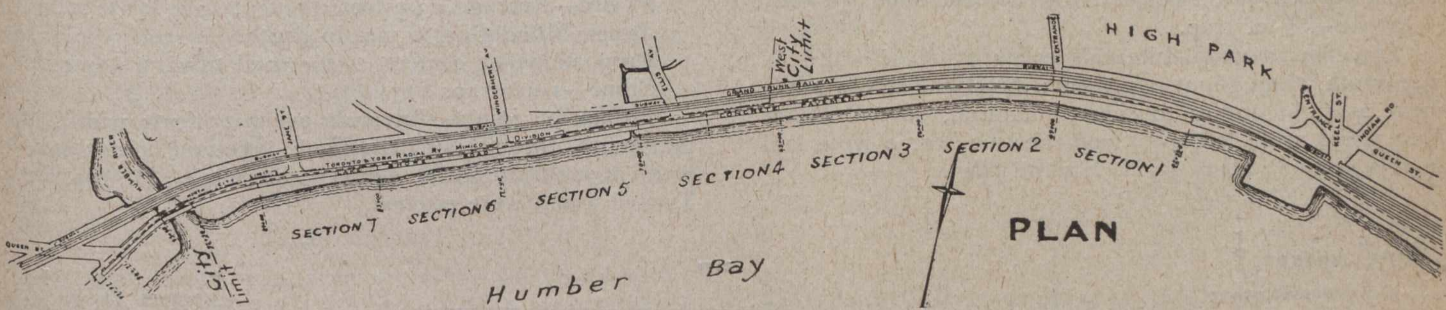
For many years a highway known as the Lake Shore Road has existed between the cities of Toronto and Hamilton in the Province of Ontario. Its name is significant of its location which is generally in close touch with the shore line of Lake Ontario between the two cities. The eastern end of the road for a distance of slightly over a mile lies within the city limits of Toronto, and it has been, and is now the duty of that municipality to maintain this portion of the road in a good state of repair. On this section the road lay at the time of the construction of the pavement, closer to the lake than at any other point throughout its entire length, its proximity to the water being merely a matter of a few feet, as is indicated on the accompanying sketch. An examination of the cross-section will indicate also the very slight difference between the water level and the road level.

In 1911 the Toronto Harbor Board was constituted with powers to formulate a comprehensive scheme for the improvement of the water front of the city. Part of the subsequent development included extensive filling along the shore, opposite the Lake Shore Road, and drastic alteration, both as to level and location of the road itself. The shore line when the work is completed will be several hundred feet farther out into the lake, and while a considerable portion of filling has already been done, no alteration has yet been made to the road. This filling has been placed at a higher level than the road, and consequently a great deal of difficulty is experienced in getting rid of water during certain seasons of the year, particularly as the drainage facilities on the road are inadequate to meet the demands. The cross-section shown indicates generally the condition brought about by the filling.

For a great many years a macadam road existed on the highway and owing to the great amount of traffic which passes over it, particularly during the summer season, and its close proximity to the water, which renders the sub-soil somewhat unstable, a great deal of difficulty has been experienced in the past in maintaining the surface in a satisfactory state of repair. The work also has been of a somewhat costly character.

*Paper contributed to the 1917-18 Transactions, American Society of Municipal Improvements.

There has been during recent years, a considerable amount of argument as to the most suitable mixture for use in concrete pavements, and the most satisfactory type of construction. In view, therefore, of the necessity which arose in 1914, of expending a large amount of money on the reconstruction of the macadam road existing at that time, it was thought that a good opportunity was presented to construct what might be called an experimental concrete pavement, particularly as it would be wiped out in the course of a few years as the harbor work developed, and failure of some of the sections would then not be a matter of great moment. An amount of money was, therefore, appropriated by the city council for the construction of a concrete pavement, and the work was divided into seven sections, each approximately 700 feet in length. The general details of these different sections are best set out in tabulated form as follows, their relative positions on the road being clearly indicated on the sketch:—



The Lake Shore Road in Toronto—Plan Showing Location of Sections

Section No. 1—Plain Concrete 1:2:5

Location—Station 42+33.8 to 49+00.
Length—666.2 feet.
Width—Average 25.2 feet.
Type—Monolithic, 6 ins. in depth.
Proportions—1 cement, 2 screened sand, 5 stone.
Stone—1-in. trap rock.
Expansion joints—Intervals, every 25 ft.; width, ½ in.; depth, throughout pavement; direction, across pavement at right angles to sides; material, Baker steel bars with ½ in. Elastite strips in some joints and ½ in. thickness of Sampson ready roofing in others.

Section No. 2—Two-course Concrete (Reinforced)

Location—Station 49+00 to Station 56+00.
Length—700 feet.
Width—Average 23.3 feet.
Type—Two-course (reinforced); first course, 4 ins. in depth; proportions, 1 cement, 2 screened sand, 5 screened gravel.
Stone—Screened gravel.
Reinforcing—Triangle mesh; second course, 2 ins. in depth; proportions, 1 cement, 1½ screened sand, 3 stone.
Stone—⅜ in. trap.
Expansion joints—Intervals every 25 ft.; width, ½ in.; depth, throughout pavement; direction, across pavement at right angles to sides; material, Baker bars, 49+00 to 49+75, Trussed Kahn bars, 50+00 to 56+00; filler, ½ in. Elastite strips.

Section No. 3—Plain Concrete, 1:1½:3

Location—Station 56+00 to 63+00.
Length—700 feet.
Width—Average 21.3 feet.

Type—Monolithic, 6 ins. in depth.

Proportions—1 cement, 1½ screened sand, 3 stone.

Stone—1-in. trap rock.

Expansion joints—Intervals every 25 feet; width, ½ in.; depth, throughout pavement; direction, across pavement at right angles to sides; materials, ½ in. Genasco strips.

Section No. 4—Two-course Concrete (Reinforced)

Location—Station 63+00 to 70+00.

Length—700 feet.

Width—Average 21.6 feet.

Type—Two-course (reinforced); first course, 4 ins. in depth; proportions, 1 cement, 2 screened sand, 5 gravel.

Stone—Screened gravel.

Reinforcing—Expanded metal; 2nd course, 2 ins. in depth; proportions, 1 cement, 1½ screened sand, 3 stone.

Stone—⅜-in. trap.

Expansion joints—Intervals every 25 feet; width ½ in.; depth, throughout pavement; direction, across pavement at right angles to sides; material, several Baker bars which were left over from Section No. 2 were laid in this section; filler, ½ in. Elastite filler.

Section No. 5—Plain Concrete

Location—Station 70+00 to 77+00.

Length—700 feet.

Width—Average 23.7 feet.

Type—Monolithic, 6 ins. in depth.

Proportions—1 cement, 2 screened sand, 4 trap.

Stone—1-in. trap.

Expansion joints—Intervals every 25 feet; width, ½ in.; depth, throughout pavement; direction, across pavement at right angles to sides; material, ½-in. Genasco strips.

Surfacing—Dolarway with stone screenings.

Section No. 6—Plain Concrete

Location—Station 77+00 to 83+00.

Length—600 feet.

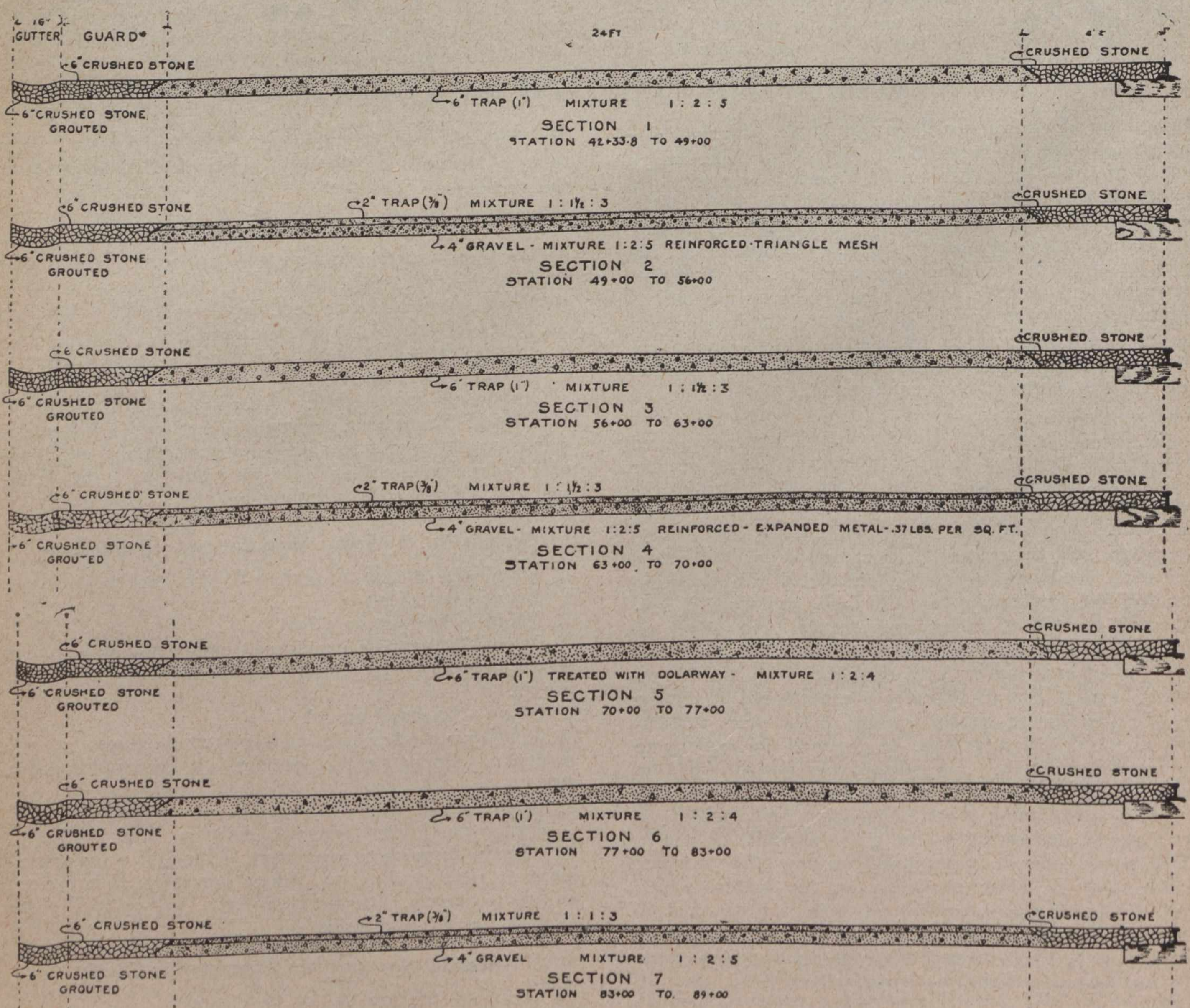
Width—Average 23.9 feet.

Type—Monolithic, 6 ins. in depth.

Proportions—1 cement, 2 screened sand, 4 stone.

Stone—1-in. trap.

Expansion joints—Intervals every 25 feet; width, ½ in.; depth, throughout pavement; direction, across pavement at right angles to sides; material, ½ in. Elastite. Twelve joints were pitched.



Cross-Sections of Concrete Pavements, Lake Shore Road

Section No. 7—Two-course Concrete

Location—Station 83+00 to 89+00.

Length—600 feet.

Width—Average 20.9 feet.

Type—Two-course; first course, 4 ins. in depth; proportions, 1 cement, 2 screened sand, 5 screened gravel. Stone—2-in. screened gravel; second course, 2 ins. in depth; proportions, 1 cement, 1 screened gravel, 3 stone.

Stone—⅜-in. trap.

Expansion joints—Intervals every 25 feet; width, ½ in.; depth, throughout pavement; direction, across pavement at right angles to sides; material, ½-in. Genasco strips.

The accompanying cross-sections, together with the above details, show clearly the various types of work constructed.

The work was commenced on May 29th, 1914, and completed September 17th, 1914, being finally opened for traffic shortly afterwards. A good deal of difficulty was experienced owing to the narrowness of the road in keeping ways open for traffic during the progress of the work, and for this reason the length of time occupied in actual construction was greater than it would otherwise have been.

Traffic Census

A traffic census was taken at various points upon the road from September 5th to September 11th (inclusive) 1915, and it was found that for 12 hours each day, from 7 a.m. to 7 p.m., an average of 1,809 vehicles passed a given point. This count included all classes of vehicles and during certain hours of the day the traffic was, of course, heavier than at others. The evening traffic, of which there is a considerable volume during the summer months, was not taken. The traffic for the last year or two has increased very materially, and the result has been to bring out certain sections in strong relief as compared to others.

The mixture which seems to have given the most satisfactory results consists of 1 part cement, 1½ of sand and 3 of 1-in. trap rock. Certain sections, also, which were reinforced have shown up to great advantage compared to those which contained no reinforcing material; the latter, in some cases being badly cracked, owing to the unstable base and aggravated surface water conditions.

Wise Use of Reinforcing

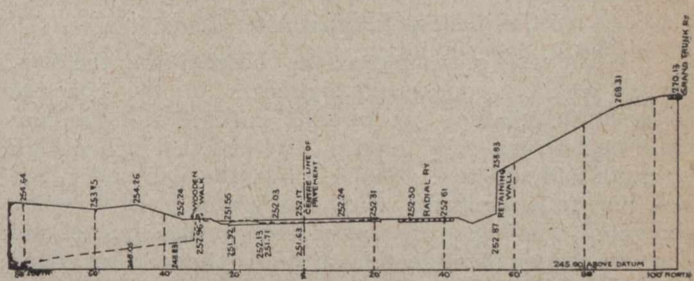
In 1912 the town of North Toronto was annexed to the city, and as very few pavements existed there at that time, the necessity for the construction of many works of this character was strongly urged. The sewerage system in the annexed district being inadequate for future needs, it was felt by the administration that it would be inadvisable to lay permanent pavements until a new system had been installed. The demand, however, for pavement accommodation was insistent, and in June, 1914, a number of concrete pavements were recommended as a temporary measure. Five of these pavements were laid in 1915 at such a grade that they could be used as part of the foundation for a future pavement of a better type.

While the information which we hoped to obtain from the construction of the pavement on the Lake Shore Road was not in any way complete, it was apparent that probably the most satisfactory mixture was 1 part of cement, 1½ of sand and 3 of stone (1-in. trap), and the work was carried out to these specifications, no reinforcing being used. All of these pavements showed longitudinal cracking the following spring, and as the reinforced portions of the Lake Shore Road showed at that time to advantage over the others, the further work done in 1916 consisted of concrete of the same mixture with the introduction of reinforcing.

These pavements have not come through the winters of 1916-17 and 1917-18 without developing any cracks. While it is not argued that reinforcing will entirely eliminate cracking, the results clearly indicate the wisdom of placing it in these later pavements. It might be added that the surface of all of these pavements up to the present time shows no sign of wear.

It is perhaps worthy of mention that several types of joints, other than the patented metal type alluded to in the Lake Shore, were tried in these pavements, and from observation of them it is evident that the most satisfactory form consists of the introduction of the ½-in. strip of manufactured filler, so placed as to permit of a small projection above the surface to allow for protection of the edges.

The information, therefore, which it was desired to gain in laying the experimental road on the Lake Shore Road, has established two very definite points in connection with the laying of concrete pavements in a climate such as that of Toronto, where the temperature has a range of 130°.



Typical Cross-Section, Lake Shore Road

Full lines show elevations of surface March 23rd, 1918, Broken lines, April 15th, 1914

They are mainly the character of mixture, and the necessity for the use of reinforcements, and in future in any case where concrete pavements are to be laid, they will follow the specifications adopted for the last-mentioned work in the northern part of the city, and will provide in addition to the necessary drainage facilities, etc., for a 1 : 1½ : 3 mixture of 1-in. trap rock and some form of reinforcement.

UNION OF CANADIAN MUNICIPALITIES

DELEGATES from all parts of Canada attended the eighteenth annual convention of the Union of Canadian Municipalities, held July 9th to 11th in Victoria, B.C. The general principles actuating the Union are summed up as follows:—

(1) The Canadian people shall not be ruled by any irresponsible monopoly.

(2) They shall not submit to methods of fraud or corruption.

(3) There must be no perpetual franchises.

(4) Our heritage of natural resources affecting municipalities must not be sold, but leased, if not publicly operated.

(5) One generation cannot legislate away the rights of another.

(6) Municipalities must control their streets.

(7) Each Canadian shall have a fair deal from all who are granted corporated or other public privileges.

(8) Some court or council must always exist, free and equipped to enforce the fair deal.

(9) The life of the poorest citizen must be made worth living through his share of the best civic conditions and services.

TEST OF FLAT SLAB FLOOR

To clear the site for the new Union Passenger Station in Chicago, Ill., the reinforced concrete building occupied by the Western Newspaper Union had to be torn down, so the opportunity was utilized to apply a test load much greater in proportion to the design load than had been used in previous tests of buildings. The tests were made in August and September, 1917, by the Engineering Experiment Station of the University of Illinois, and the tests and their results are described in Bulletin No. 106 published by the University.

A load of 913 pounds per square foot was applied over four panels of a four-way reinforced concrete flat slab floor. The building was nine years old at the time of the test. The test was carried far enough to give stresses in the reinforcing bars and concrete markedly higher than had been obtained in other building tests. As the result of the tests the following deductions have been drawn by the engineering staff of the University:—

Conclusions Formed After Tests

(1) The tests of samples of the concrete from the slab, as well as the hardness and toughness of the concrete observed in breaking up the slab, indicate that the concrete was of unusually good quality and that it had high strength and stiffness. The action of the slab under load was that to be expected with high-grade, well-seasoned concrete. The effect of time on the stresses in steel and concrete and on the deflection of the slab under a sustained load was slight, even over a period of 66 hours under the maximum load of 913 lbs. per square foot,—conditions which would not exist at an early age of concrete. Upon removal of load, the recovery in deflection at the centres of the panels was about 75 per cent. of that under load, and at other points generally more; the recovery in strains in steel and concrete was as large.

(2) The position of the important cracks on both upper and lower sides of the slab may be expected to indicate the region of high tensile stresses in the reinforcing bars; it is also an indication of the general action of the slab in flexure. The cracks on the upper side at the load of 913 lbs. per square foot opened to a width of 0.02 to 0.06 in.; those on the lower side were not so wide. Upon removal of the load the cracks closed, leaving the surfaces of the slab with the appearance which they had before the load was applied.

Stresses in Reinforcing Bars

(3) For reinforcing bars in the upper side of the slab in the regions of negative moment, the stresses in bars of diagonal bands were greater than those in bars of rectangular bands, a stress of 57,300 lbs. per square inch being observed in a diagonal bar and one of 42,000 lbs. per square inch in a rectangular bar at the maximum load. Stresses were found in both rectangular and diagonal bars at the columns bordering the loaded area nearly as great as those at corresponding points at the central column. Stresses of some magnitude were found in bars outside the loaded area. The stresses given do not include the stress due to the load of the slab itself.

(4) For reinforcing bars in the lower side of the slab in the regions of positive moment, the stresses in bars of rectangular bands were greater than those in bars of diagonal bands even though the former were farther above the lower surface of the slab than the latter; in the one apparent exception, the presence of laps doubled the usual number of bars. At the maximum load, stresses of 24,000 to 30,000 lbs. per square inch were observed in bars of rectangular bands and of 20,000 to 24,000 lbs. per square inch in bars of diagonal bands. A stress of 15,600 lbs. per square inch was observed in a bar outside the loaded area at the edge of a rectangular band.

(5) On the upper surface of the slab the greatest compressive strains were found at gauge lines along the inner panel edges midway between columns, ranging from 0.0009 to nearly 0.001 in. per inch at the maximum load; at the centres of the panels values about half as great were found. On the lower surface the greatest strains were found at the middle column; these ranged from 0.0012 to 0.0016 in. per inch, values which are as great as the strains found at failure in the tests of the concrete prisms cut from the slab and as great as are ordinarily found in compression tests of concrete at the ultimate load. In some places there was shipping and spalling of the concrete. It is evident that the action of the surrounding concrete assisted in preventing failure. At gauge lines crossing the inner panel edges high compressive strains also were found, even though there was no tension reinforcement in the upper side of the slab in this region. It may be noted that the maximum compressive strains on the upper surface of the slab were one-half to three-fifths those found on the lower surface. The intensity of the strains at various points along sections of positive moment and negative moment will give some measure of the distribution of intensity of moments along those sections.

Comparison With Joint Committee Report

(6) The observed stresses in the reinforcing bars accounted for about 90 per cent. of the analytical negative moment and about 70 per cent. of the analytical positive moment, as given by the methods used. It should be noted that the observed stresses used are average stresses over the gauge length, and the stress at a crack may be expected to be greater than the average over the gauge length. It seems probable that tensile resistance of the concrete contributed to the resistance of the slab, particularly in the sections of positive moment and in regions near the edges of the loaded area. A similar influence of the tensile resistance of concrete, when the stresses in the steel are well below its yield point, has been observed in numerous beam tests. That the tensile resistance of the concrete contributed to the resisting moment of the slab in the test should not be taken to mean that it will be effective in resisting moment when the ultimate load is reached. It may be noted also that the sum of the positive and negative moments accounted for by the measured stresses in the reinforcing bars has almost the same value as the sum of the positive and negative moments recommended by the Joint Committee on Concrete and Reinforced Concrete, and that the negative moments so accounted for are about 113 per cent. and the positive moments about 73 per cent. of the moments recommended by this committee. In making a comparison with methods used in designing, it should be borne in mind that the principal maximum stresses were from 15 to 25 per cent. greater than the average stresses which were used in computing the resisting moments accounted for by the stresses in the bars; in designing, a uniform stress over the section is assumed.

"Should Give Added Confidence"

Although the arrangement of bars was not as recommended, the amount of reinforcement for negative moment, considering all available bars over the area used, was as much as that required for the negative moments recommended by the Joint Committee on Concrete and Reinforced Concrete, even though the slab was thinner than recommended for ordinary concrete. The amount of reinforcement for positive moment was more than 50 per cent. greater than that required for the positive moments recommended by the committee. Although the nominal thickness of the slab was less than that required by building regulations, it fulfilled the provisions of the committee for bending moments and working stresses for concrete of a test strength of 3,000 pounds per square inch.

(7) The action of the floor slab under test should give added confidence in the suitability and reliability of the flat slab as a load-carrying structure.

EARTH PRESSURES*

By Leo Hudson

Consulting Engineer, Pittsburg, Pa.

THE purpose of this paper is the epitomizing of the voluminous literature reviewed on the subject of "Earth Pressures" with the hope that it will fill the same purpose in other offices which it has served to fill in my office. In offering this addition to the already extensive literature we hope to establish that rule which will be of the easiest application, which may be acceptable to the busy engineer and at the same time be of value to the young engineer through the light of his own understanding.

The fundamentals underlying this work are not new. The wedge theory is that of Coulomb. We have tried to make the application prove in mathematical agreement with Rankine's formula. From this we have derived a formula which seems to be of easier application, and also a graphical solution. We have proven our formula both analytically and graphically. We have used freely the work of J. Romilly Allen, A.I.C.E., as published in Van Nostrand's Magazine, Vol. XVII., p. 155. Also we have used certain features of the notes as prepared by M. Monduit and taught in the Ecole des Beaux Arts, Paris. We have purposely kept away from the refinements of later writers because we would find the maximum conditions and design to meet them.

Behind each linear foot of wall holding an earth fill there are many sliding wedges which may be assumed, but there is only one wedge which will produce the maximum force and it is this wedge and its force for which the wall must be designed. In turn, this maximum resultant force may be assumed to act in many directions and at many points, but it is the direction and point of application producing the maximum force in which we are especially interested. These functions of the resultant force, namely, its magnitude, direction and point of application are sought out in terms of their maximum effects and the formulæ are based on these limits.

We have left the question of the friction of the earth on the back of the wall and the friction of the bottom of the wall on the ground to the design of the wall to meet the conditions. Also, in gravity walls with the back making an angle of greater than 90 degrees with the horizontal, instead of adding the weight of the materials between the back of the wall and a vertical line we might step the wall so that this would become an aid instead of an extra weight. In a smooth-backed reinforced wall, however, this additional weight must be added as indicated in the work to follow.

*Abstracted from Transactions of the American Society of Municipal Improvements.

To determine the effect of earth thrust against the back of a wall it is necessary to determine:

(1) The magnitude of the pressure.
(2) The point of application.
(3) The line of action.

These elements may vary greatly but there is a time when their action amounts to a maximum. It is this maximum condition against which we should design. It is best to neglect the cohesion of the earth particles to each other except in as much as this cohesion affects the "angle of friction," as cohesion of earth might be rendered ineffective by shock, and to consider it away from the line of rupture would be giving to the earth a slight tensile strength which might not, under certain conditions, exist.

It is at once apparent that in a mass of earth behind a wall there is one wedge which will cause the maximum thrust against the wall. One feature of our problem is the determination of this wedge. After this wedge which exerts (1) the maximum pressure is determined, we are confronted with the problem of (2) the point of application of its resultant force. Opinion differs widely as to

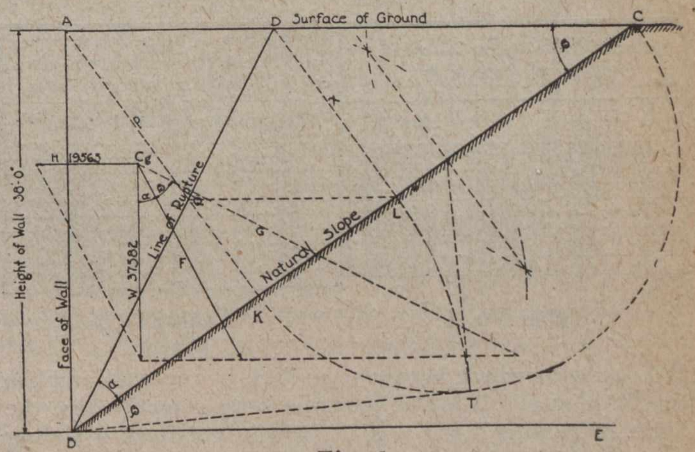


Fig. 1

this; some engineers taking one-third up the wall from the base, some taking four-tenths up from the base, and others points between these two points. If the material behind the wall were liquid then the resultant pressure would be one-third up from the base; if the material were solid then the resultant pressure would be one-half way up from the bottom. For earth, which is never either liquid or solid, it would seem that one-half the difference between one-third and one-half should be added to one-third, making the point of application at .415 up from the base or practically four-tenths up. (.50 — .33 = .17; .17 ÷ 2 = .085; .085 + 3.3 = .415). As to (3) the direction of the line of action, the direction may be horizontal or make an angle with the horizontal. For an horizontal surface we would take the direction of the resultant as horizontal and for sloping surfaces we would still take it as horizontal and add the weight of the material above the horizontal top as surcharge (see Fig. 11). Anyway, the horizontal direction requires the heavier wall and as the material might produce a resultant which would act horizontally we would take that direction. Then, there is a wedge which will exert a maximum resultant pressure, said resultant acting at a point four-tenths of the height of the wall up from its base, and acting in a horizontal direction.

Now the problem is to determine this wedge which will exert the maximum pressure. All earthy material, if poured vertically on a horizontal plane, will form a surface slope making an angle with the horizontal. This angle is called the "angle of repose." Each particle on the slope is held in equilibrium by the force of gravity, due to its weight, and by friction. The tangent of the angle of repose is the "coefficient of friction" of the material. We all have noticed in a bank about to cave, that there first becomes a large crack near the edge of the bank and sometimes this crack partially develops before the cave takes place. Then back of this large crack there is a smaller one, then behind this still a smaller one, and so on until finally there is a mere "hair crack" almost twice as far back from the edge of the bank as the first large crack. This material between the first large crack and the edge of the bank is known as "the sliding wedge." And it is this wedge which we will undertake to prove, of all wedges, will exert the maximum thrust against the wall. The plane of the first large crack is called the "plane of rupture."

Take Fig. 1. Let AB be the face of a vertical wall, let BE be the horizontal, let AC be the surface of an earth fill placed behind the wall. Then let BC be the line of repose of the material making the angle b with the horizontal. The angle b is the "angle of repose." The line

Value of Angle α For Different Angles of Repose β			
β	α	tan α	tan² α
10°	40°	0.83910	0.70409
15°	37½°	0.76733	0.58879
20°	35°	0.70021	0.49029
25°	32½°	0.63707	0.40586
30°	30°	0.57735	0.33333
35°	27½°	0.52057	0.27099
40°	25°	0.46631	0.21745
45°	22½	0.41421	0.17157
50°	20	0.36397	0.13348

Table No. 1

Weight of Sliding Wedge Behind Vertical Wall When Surface is Horizontal. Angle of Repose 35° Weight of Material 100# per cu. ft.		
Height of Wall	Distance of line of Rupture from Top of Wall.	Weight of Sliding Wedge.
5'	2.6'	650#
10'	5.2'	2600#
15'	7.8'	5850#
20'	10.4'	10400#
25'	13.0'	16250#
30'	15.6'	23400#
35'	18.2'	31850#
40'	20.8'	41600#
45'	23.4'	52650#
50'	26.0'	65000#

Table No. 2

BD lies somewhere between the face of the wall and the line of repose. It is our purpose to determine where the "line of rupture" BD will be, which will form a sliding wedge which will exert the maximum pressure against the wall. The resultant pressure of the sliding wedge ABD on BD makes an angle to the normal to BD equal to the coefficient of friction of the material, which angle is called the "angle of internal friction" and may differ materially from the "angle of repose." The friction of the "sliding wedge" is a different friction force from the surface-forming friction, one being due to a sliding action, the other being due to a rolling action. For loose, earthy material, it is safer and at the same time not too safe to assume that the "angle of internal friction" is equal to the angle of repose and therefore to take the resultant of the "sliding wedge" as acting with the normal to the line BD decreased by the "angle of repose." (For a liquid there would be no angle of repose and no line of rupture, and if W is the weight of a cubic foot of the liquid, h is the height at any point on the wall, and P is the horizontal pressure, then $P = \frac{1}{2} Wh^2$.)

To Determine the Line of Rupture

Again in Fig. 1 let AB be the back of a retaining wall, BE the horizontal, BC the line of repose making the angle b with the horizontal.

Then, from A, draw AK perpendicular to BC; bisect KC; with the centre point of CK as centre draw a semicircle through K and C; from B draw a line tangent to the semi-circle; then let T be the point of tangency; with B as a centre and BT as radius, describe arc TL cutting BC at L; from L draw LD perpendicular to BC, cutting AC at D; connect D and B. The line BD is the "line of rupture" and divides ABC into ABD and DBC. The wedge ABD is the "sliding wedge" which will exert the maximum pressure against the wall AB.

To Determine the Pressure Which ABD Would Exert Against AB

The sliding wedge is held in equilibrium by gravity (due to its weight), friction which it makes on the line BD and the wall. To determine the pressure against the

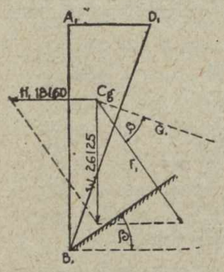


Fig. 2

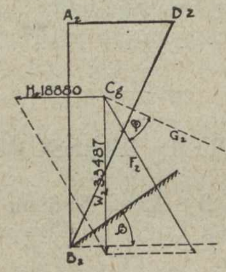


Fig. 3

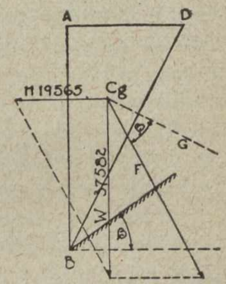


Fig. 4

wall, exerted by the wedge ABD, consider said wedge acting on one linear foot of wall; find centre of gravity of wedge. (The centre of gravity is at the intersection of lines drawn from any two angles to the centre of the opposite sides.) From the centre of gravity drop a line which represents the weight of the wedge one foot thick, W; from the centre of gravity draw a line normal to BD; set off from this line the angle of internal friction b; which gives the line F; divide W into its two components, one acting on BD along the line F; and acting horizontally against the wall as H. By drawing W to scale and measuring H to the same scale, we find the magnitude of the force H against the wall which acts at a point

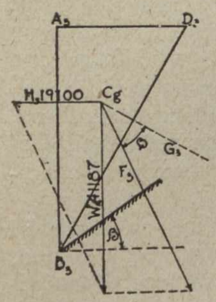


Fig. 5

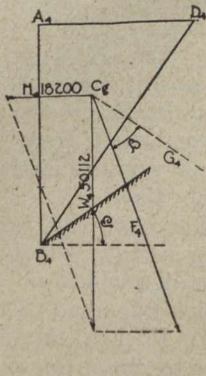


Fig. 6

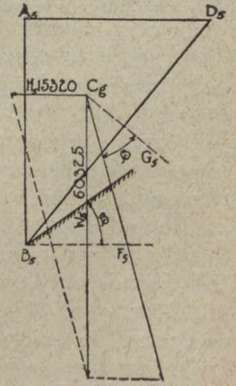


Fig. 7

0.4 of the height of the wall up from the base. It is apparent that the angle which W makes with F is equal to a.

Then: $H = W \tan a.$ (I)

Then again: H attains its maximum value when Area ABD tan a is greatest.

Let $p = AK$; $q = BC$; $b =$ angle ACB; which quantities are constant.

$x = DL$; $a =$ angle DBC; which quantities are variables.

Now: Area ABD $\tan a = (\frac{1}{2}pq - \frac{1}{2}xq) \tan a.$

$$= \frac{1}{2}q \left(p - x \left(\frac{x}{q - x \cot b}\right)\right)$$

$$= \frac{1}{2}q \left(\frac{px - x^2}{q - x \cot b}\right)$$

Differentiating the quantity $\frac{px - x^2}{q - x \cot b}$ and putting the value obtained equal to o for a maximum, there results the following equation:

$$(q - x \cot B)\ \ (p - 2x) - \ (px - x^2)\ \ (-\cot b) = o.$$

$$\text{or } x^2 \cot b - 2qx = -pq \qquad \text{(II.)}$$

Putting this equation into another form

$$pq - qx = qx - x^2 \cot b$$
$$= x\,(q - x \cot b)$$
$$= x.BL$$

Area ABC — DBC = Area DBL

Area ABD = Area DBL (III.)

which equality expresses the only condition necessary in order that ABD may be the "Prism of Maximum Earth Thrust," and BD the "Plane of Rupture."

The actual amount of earth thrust is found thus:

From Eq. I.: H = (weight of Prism ABD) tan. a

= (weight of Prism DBL) tan. a

$$= w.\tfrac{1}{2}DL.\ \frac{BL.}{BL}$$

$$H = \tfrac{1}{2}\,wx^2 \qquad \text{(IV.)}$$

Where H = maximum horizontal earth thrust per ft. forward:

W = weight of 1 cu. ft. earth; x = DL; x may either be found by solving equation (II.) or may thus be expressed in terms of known quantities.

$$x = DL = LC \tan b = (BC - BL) \tan b$$

$$x = (BC - \sqrt{BC.\ BK})\ \tan.\ b. \qquad \text{(V.)}$$

The above formula applies equally well whether the surface behind the wall be horizontal or not, when the back of the wall is vertical of height h, and the surface of the bank horizontal.

$$H = \tfrac{1}{2}\,wh^2\,\frac{1 - \sin b}{1 + \sin b} \qquad \text{(VI.)}$$ which is Rankine's formula.

Fig. 8

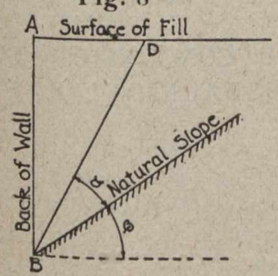
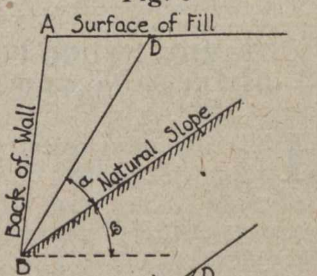


Fig. 9



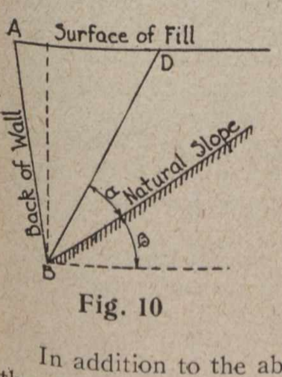


Fig. 10

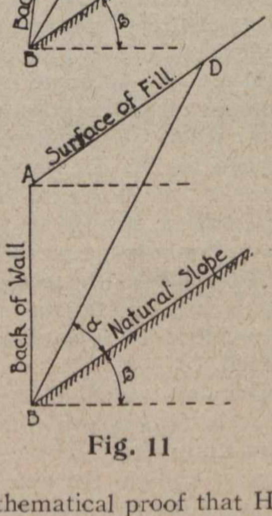


Fig. 11

In addition to the above mathematical proof that H is the maximum for the wedge formed by the line BD we give the following graphical demonstration:

Let Figs. 2 to 7 represent a wall, the same as in Fig. 1, where the natural slope makes an angle of 35° with the horizontal. In Fig. 2 assume a line B_1 D_1 as the line of rupture; in Fig. 3 assume a line B_2 D_2 as the line of rupture; in Fig. 4 make BD the line of rupture, the same as in Fig. 1; in Fig. 5 assume a line B_3 D_3 as the line of rupture; and so on. In each case these figures show H as graphically determined, to be greatest in Fig. 4 which is the H we are seeking.

From the above discussion we draw the following general conclusions:—

First: For a vertical backed wall and horizontal surface on fill, as in Fig. 8, the line of rupture bisects the angle between the back of the wall and the line of natural slope. The resultant pressure H against the wall is the weight of the triangle of earth so obtained multiplied by the tangent of one-half the angle between the back of the wall and the line of natural slope, which is the angle between the line of rupture and the line of natural slope.

Second: For a wall, the back of which makes an angle of less than 90 degrees with the horizontal and the surface of the fill horizontal, the same rule applies as in the first case. See Fig. 9.

Third: For a wall, the back of which makes an angle of more than 90 degrees with the horizontal and the surface of the fill horizontal, proceed exactly as in the first case and add to the force H obtained the weight of the prism of earth between the back of the wall and the vertical line passing through the base. See Fig. 10.

Value of H for Given Heights Where Earth Weight is 100# Angle of Repose 35° Angle of Internal Friction 35°	
Height of Wall	Value of H
5'	338#
10'	1353#
15'	3045#
20'	5414#
25'	8459#
30'	12181#
35'	16580#
40'	21655#
45'	27408#
50'	33837#

Table No. 3

Fourth: For a wall holding a fill with a sloping surface, as in Fig. 11, proceed as in Fig. 8 and continue the line to the surface. Then find the centre of gravity of the entire wedge between the wall and BD and divide into its two components as in all other cases.

When back of wall is vertical and surface of fill horizontal, weight of fill taken at the usual figure of 100 pounds per cubic foot, then H is equal in pounds to the height of the wall times tan. a, times one-half height, times 100, times tan. a;

$$\text{or } H = [(h.\ \tan.\ a)\ \tfrac{1}{2}h.\ 100]\ \tan.\ a$$
$$\text{or } H = (\tfrac{1}{2}h^2\ \tan.^2\ a)\ 100$$

Angle a, being equal to 90 degrees less angle of repose divided by 2.

Weight of earthy materials in pounds per cubic foot: sand, 105; gravel, 135; gravelly clay, 130; ordinary earth, 100; hard pan, 130.

Angles of repose degrees: Coal, 54; bank sand, 54; earth, 35 to 48; quick sand, 37; clay, 42; cinders, 25; gravel, ½-in., 25; gravel, ¼-in., 19; fine sand, 10.

Ordinary conditions: We take earth at 100 lb. per cubic foot with angle of repose of 35 degrees and angle of internal friction of 35 degrees.

Example

The following example will demonstrate the above principles in terms of the working data given:—

What is the earth pressure behind a vertical wall which has a height of 38 feet, the surface of the fill horizontal, angle of repose 35 degrees, weight of fill 100 pounds per cubic foot?

Answer: Here a = 90 degrees less 35 degrees, divided by 2 or 27½ degrees. Then $\tan^2 a = 0.27099$ (see Table 1). Now $H = (\tfrac{1}{2}h^2 \tan^2 a)\ 100 = (722 \times 0.27099)\ 100 =$ 19,565 pounds.

Problem: Under the above conditions with a wall 25 feet high, what would be the resultant pressure in pounds?

Answer: See Table No. 3—8,459 pounds.

"MUNICIPAL IMPROVEMENTS" CONVENTION

At the last annual meeting of the American Society of Municipal Improvements, it was left to the executive committee to decide upon the place of the next convention; the committee has now decided that it shall be held October 2nd, 3rd and 4th, 1918, at Buffalo, N.Y. It is necessary in these days of railroad congestion to reduce travel to a minimum, and Buffalo is very near the centre of membership of the society, especially of the district in which the membership is most concentrated. The society was organized in October, 1894, in Buffalo.

The usual time of the convention has been shortened by eliminating purely entertainment features so as to reduce hotel bills, but arrangements will be made for members to see all the points of engineering interest in Buffalo. The executive committee announce that the high level of previous conventions will be sustained so far as papers and discussions are concerned. Already several most valuable contributions to municipal engineering practice have been promised.

Considering the fact that the 1917 convention was omitted, the loss of membership in the past two years has been surprisingly small, says the executive committee. The number of new members is much less than usual, however, as the conventions have been the principal source for them; the number of members has been reduced by about eighty, so members are requested to bring new prospects with them to the convention, in order that the loss may be made up. George H. Norton, city engineer of Buffalo, is chairman of the committee on convention arrangements.

ACID PROCESS FOR SEWAGE DISPOSAL

In a recent number of "Metallurgical and Chemical Engineering," a process for sewage treatment and disposal, discovered and patented by George W. Miles, was described, from which the following information is abstracted:—

Receiving tanks are constructed at the sewer outfall. At these tanks a small part of the sewage is pumped through an absorption tower with a counter current of SO_2. The sewage that has been saturated with SO_2 is mixed with the rest of the sewage at a point sufficiently distant from the mouth to ensure thorough mixing. All the sewage then runs into a settling tank and from there into a second settling tank. The effluent flows from the second tank. The tanks must be large enough to provide four hours' sedimentation in all. During this period, 98 per cent. of the solids recoverable by settling are precipitated. After one set of settling tanks has been operated for about a week, the sewage is switched to another set of tanks. The sludge, which contains from 80 to 92 per cent. of water, is then pumped from the first set of tanks. The sludge is not offensive, and there is no putrefaction. The sludge is pressed and then dried under careful temperature regulation to prevent burning.

About one ton of SO_2 is required per million gallons of sewage, equivalent to 1,000 lbs. of sulphur or 2,240 lbs. of pyrites. Nitre cake and SO_2 can also be used, in the proportion of 3.82 tons of nitre cake and 200 lbs. SO_2 per million gallons of sewage. The amount of sludge varies, of course, with the character of the sewage. With average city sewage there will be about 2,000 lbs. of sludge per million gallons, containing 400 lbs. of grease. The grease is extracted from the sludge and the remainder can be used for fertilizer. The grease-free sludge contains about 4.5 per cent. of NH_3 and between 1 and 2 per cent. of P_2O_5.

The cost of the process and the value of the products have both been affected by war conditions. With pre-war prices, the cost of the acid treatment would not exceed $9 per million gallons of sewage. The value of the grease recovered would be $16 and of the sludge as fertilizer, $10.20 per million gallons, making $26.20 in all, and leaving $17.20 to cover the cost of drying and extracing the grease from the sludge. With prices as of January, 1918, the cost of treatment would be $20, and the value of the grease and fertilizer, $90.94 per million gallons, leaving $70.94 for drying and extracting grease.

REPORT ON CONCRETE SHIPS

At the suggestion of Senator Smith, of Arizona, the Government Printing Office at Washington, D.C., has issued a pamphlet known as Senate Document No. 239, "Construction of Concrete Ships," containing report dated April 5th, 1918, by R. J. Wigg, chief engineer of the Department of Concrete Ship Construction of the Emergency Fleet Corporation, to Edward N. Hurley, chairman of the Corporation; estimate of appropriation covering an emergency fund for concrete ships made by Mr. Hurley under date of April 9th, addressed to the Secretary of the Treasury; report on the construction, use and advantages of concrete ships by Benjamin A. Howes, addressed to the chairman of the Committee on Commerce of the U.S. Senate, April 27th; report by Roy H. Robinson regarding the construction and advantages of concrete ships and commenting upon Mr. Howes' report, addressed to the chairman of the Committee on Commerce, May 18th.

ONTARIO HYDRO-ELECTRIC COMMISSION TO DEVELOP POWER IN NORTHERN ONTARIO

In a statement recently made to the representatives of the municipalities of London, Ont., and surrounding district, Sir Adam Beck announced that the Ontario government had passed an order-in-council authorizing the development of 130,000 horse-power at Nipigon, New Ontario. The ultimate object is to develop the mineral resources of that portion of Ontario.

Sir Adam also informed the meeting that he had telegraphed to W. T. Sims, chairman of the Power Committee of the House of Representatives at Washington, D.C., that by the fall of 1919 the Ontario Hydro-Electric Power Commission would be in a position to supply the United States with 200,000 horse-power and would also be able to increase the supply by 30,000 horse-power per month until a maximum of 300,000 horse-power was reached.

He also intimated that three other orders-in-council had recently been passed by the Ontario Government, one authorizing the increase of electrical equipment at Niagara Falls to the extent of half a million dollars, and others authorizing the development of from 25,000 to 30,000 horse-power at High Falls on the Rideau River and on the St. Lawrence River between Cornwall and Brockville.

The Boston and Maine Railroad has decided to use its old ties for fuel. This is a timely change from the familiar practice of making bonfires alongside the tracks. The Boston and Maine management proposes to use the old ties instead of coal next winter in stations and workmen's shanties.

MINNEAPOLIS CITY WATERWORKS, DEPARTMENT OF PURIFICATION*

By F. W. Cappelen
City Engineer, Minneapolis

THE purification plant for the Minneapolis Water Department was put in operation January 10th, 1913. The plant consisted of twelve filter beds, each of four million gallons maximum capacity, with two coagulation chambers, each of 1,300,000 gallons capacity.

During the year it was found that the coagulation basins were not large enough, and also that the filter capacity had to be increased.

This work was completed, in 1915, by adding four filters, each of four million gallons maximum capacity, and two coagulation basins, each of 1,500,000 gallons capacity.

In 1917 we again commenced construction of eight filters, giving us a total of ninety-six million gallons which will be in operation this fall. But the complete report for 1917 is not yet out, so the complete laboratory operation, and cost data for the four years that the filtration plant has been in service will be found in the following tables, making the comparison possible for 1913, 1914, 1915 and 1916, at a glance.

Table No. 1—Shows amount of water filtered; rate of filtration; wash water; chemicals used; fuel and electric energy used.

Table No. 2—Cost data.

Table No. 3—New construction cost, not chargeable to operation.

Table No. 4—Routine chemical laboratory tests; odor; turbidity; color; alkalinity, etc.

Table No. 5—Sanitary chemical data.

Table No. 6—Analysis of mineral residues.

Table No. 7—Bacteriological data.

Table No. 8—Bacteriological data; B. coli determinations.

Table No. 9—Microscopical examinations.

Table No. 10—Typhoid fever statistics.

Table No. 11—Comparison of color in river water with precipitation data.

Table No. 12—Relation of amount of water filtered to precipitation and temperature data.

Table No. 13—Reduction in color per grain of alum.

Chart No. 1—Typhoid fever chart.

*Abstracted from paper read before the American Society of Municipal Improvements.

Typhoid fever statistics for Minneapolis for the years 1900 to 1916, inclusive, will be found in Table No. 10, while the accompanying Chart No. 1 shows diagrammatically the typhoid death rate per 100,000 for the same period. It is interesting to note that the typhoid death rate has remained practically the same since 1911, the year following the installation of the hypochlorite of lime treatment of the city water supply, until the year 1915, when it dropped to 7 per 100,000. In 1916 there was a further drop in the typhoid death rate to 4.7 per 100,000, the lowest for Minneapolis of which there is any record, and one of the lowest of any city in the United States.

Table 1—Filtration Data

	Filters: Rate of filtration Mill. gals. 24 hours	Filters: Number of filters in service	Filters: Period between washings Hours	Filters: Number of filters washed	Filters: Loss of Head Feet	Water Filtered Gallons: Total	Water Filtered Gallons: Net	Wash Water: Gallons	Wash Water: Per Cent	Coagulant: Pounds	Coagulant: Grains per gallon	Lime: Pounds	Lime: Grains per gallon	Chlorine: Pounds	Chlorine: Parts per million	Fuel used: Pounds	Electric Energy Used: K.W.H.
1913																	
Max. daily						45,817,000	44,452,000	1,218,000	6.3	34,417	8.11						
Min. daily						14,704,000	14,192,000	170,000	0.7	1,396	0.39						
Aver. daily	3¼	9	13½			25,889,000	25,114,000	775,000	3.0	12,820	3.47						
Total						9,294,198,000	9,015,907,000	278,291,000		4,602,517				b 210,652		625,610	286,362
1914																	
Max. daily						44,949,000	42,462,000	2,860,000	d 9.8	39,750	9.36						
Min. daily						10,256,000	9,682,000	0	0.0	1,400	0.50						
Aver. daily	3¼	9	24½			25,139,000	24,129,000	1,010,000	4.0	11,175	3.11						
Total						9,175,840,000	8,806,942,000	368,898,000		4,078,783				b 199,544		734,621	245,210
1915																	
Max. daily						36,806,000	35,933,000	1,728,000	7.1	24,700	5.70						
Min. daily						17,800,000	17,254,000	186,000	0.8	2,200	0.64						
Aver. daily	2¼	13	37¼			25,687,000	24,743,000	944,000	3.7	9,641	2.63			b 290 c 62	0.3		
Total						9,375,681,000	9,031,060,000	344,621,000		3,518,974		a 239,658		b 104,518 c 1,600		870,769	268,180
1916																	
Max. daily						58,929,000	57,237,000	2,208,000	6.1	37,200	5.81						
Min. daily						19,827,000	18,809,000	0	0.0	2,850	0.80						
Aver. daily	2.1	15	44¾			29,289,500	28,319,800	969,700	3.3	10,636	2.54			c 75¼	0.32		
Total						10,719,959,000	10,365,036,000	354,923,000		3,892,780				c 27,541		1,010,930	295,940

Notes: a, Lime used in water softening experiments. b, Hypochlorite of lime. c, Liquid chlorine. d, High wash-water due to micro-organisms.

Table 2—Cost Data

	Cost of Purification: Coagulant	Lime	Chlorine	Fuel	Electric energy	Wash Water	Supervision and Office: Wages	Supervision and Office: Supplies	Laboratory: Wages	Laboratory: Supplies	Operation: Wages	Operation: Supplies	New machinery and equipment: Wages	New machinery and equipment: Supplies	Total purification cost: Wages	Total purification cost: Supplies	Maintenance and repairs: Wages	Maintenance and repairs: Supplies	Total Cost: Wages	Total Cost: Supplies	Credits	Interest and depreciation	Total net cost
1913																							
Total	43,263.65		3328.24	1567.26	3170.74	5566.65	2960.00		3480.00		17,098.50	490.124							23,538.50	61,797.78			85,336.28
Per mill. gals.	4.66		0.36	0.17	0.34	0.60	0.32		0.38		1.84	0.53							2.53	6.65			9.18
1914																							
Total	37,746.62		3165.87	1835.86	3060.85	7377.96	3626.64	120.23	3480.00	1067.49	18,125.35	1234.96	126.13	1876.48	25,350.12	57,496.32	1811.75	2252.84	27,169.87	59,739.16	691.65		86,267.38
Per mill. gals.	4.11		0.35	0.20	0.33	0.80	0.40	0.01	0.38	0.12	1.98	0.14	0.01	0.20	2.76	6.26	0.20	0.25	2.96	6.51	0.07		9.40
1915																							
Total	32,501.92	913.41	1922.38	1930.10	3016.56	6892.42	3960.00	59.16	3480.00	937.76	17,646.52	998.80	11.62	752.06	25,098.14	49,924.57	2314.11	1282.68	27,412.25	51,207.25	0.00		78,619.50
Per mill. gals.	3.47	0.10	0.21	0.21	0.32	0.74	0.42	0.01	0.37	0.10	1.88	0.11	0.00	0.08	2.68	5.32	0.25	0.14	2.92	5.46	0.00		8.38
1916																							
Total	45,545.52		2946.88	2432.49	3091.14	7098.40	3960.00	219.75	3606.87	1009.01	18,085.27	1162.02	82.50	490.60	25,744.64	64,005.81	4146.84	1577.31	29,891.48	65,583.12	0.00		95,474.60
Per mill. gals.	4.25		0.27	0.23	0.29	0.66	0.37	0.02	0.34	0.09	1.69	0.11	0.01	0.05	2.40	5.97	0.39	0.15	2.79	6.12	0.00		

Table 3—New Construction Cost

	Filtration Plant Railway: Construction Wages	Construction Supplies	Maintenance Wages	Maintenance Supplies	Operation Wages	Operation Supplies	Int. and Deprec.	Total Cost	New Construction: Wages	New Construction: Supplies	Reservoir Boulevard: Wages	Reservoir Boulevard: Supplies	Old Reservoir Expense: Wages	Old Reservoir Expense: Supplies	Public Safety: Wages	Public Safety: Supplies	Total Cost: Wages	Total Cost: Supplies	Credits	Total Net Cost
1913																				
1914									84.80	1028.58							84.80	1028.58		1113.38
1915									372.75	2252.50	621.51	449.19	67.50	22.50			1061.76	2724.19		3785.95
1916	42.50	0.00									758.44	726.53					800.94	726.53		

The relation between the rainfall on the Mississippi watershed above Minneapolis, and the color of the river water at Minneapolis, is shown in Table No. 11. This relation is masked somewhat by the storage of the flood waters in the large reservoirs controlled by the United States government, and also influenced by the intensity of the precipitation during short intervals of time. It may be stated in general, from the data at hand, that the cumulative effect of the rainfall upon the color for the succeeding months is apparent and that a decided increase in color of the river water at Minneapolis follows in from fifteen to thirty days a heavy precipitation on the Mississippi watershed above Minneapolis.

Table No. 12 shows the relation of the amount of water filtered each month of the years 1913, 1914, 1915 and 1916, to the total monthly precipitation and average temperature data in Minneapolis for each month as supplied by the United States Weather Bureau.

Table No. 13 indicates the added efficiency of the two additional coagulation basins in reducing the amount of alum used. The parts per million of color removed by one grain of alum is shown for each month of the four years, as compared with the number of coagulation basins in service.

Operation and Cost Data

Filters.—Table 1 shows that the greatest total amount of water filtered during any one year was in 1916, with a total of 10,719,959,000 gallons, an increase of 1,344,278,000 gallons over 1915. The average daily amount of water filtered during 1916 was 29,289,500 gallons, an increase of 3,602,500 gallons per day over the 1915 average. The maximum amount of water fil-

Table 4—Summary of Routine Chemical Laboratory Tests

	Descriptive				Parts per million.																									
	Odor		Temp. C°		Turbidity			Color			Alkalinity as $CaCO_3$ Phenolphthalein			Alkalinity as $CaCO_3$ Erythrosine			Incrustants as $CaCO_3$			Total Hardness as $CaCO_3$			Magnesium as $CaCO_3$			Carbonic Acid as CO_2			Residual Chlorine	
	S.B.	C.E.	S.B.	C.E.	R.W.	S.B.	C.E.	R.W.	S.B.	C.E.	R.W.	S.B.	C.E.	R.W.	S.B.	C.E.	R.W.	S.B.	C.E.	R.W.	S.B.	C.E.	R.W.	S.B.	C.E.	R.W.	S.B.	C.E.	C.E.	D.M.
1913																														
Max. daily	M.	0	27.5	27.5	30	20	0	108	102	43				198	224	219				208	248	252				5	9.5	36.	.180	—
Min. daily	0	0	0.8	1.0	10	10	0	38	11	4				129	106	76				135	118	114				0	0	1.5	.000	—
Aver. daily	0	0	11.0	11.4	18	8	0	59	45	18				153	163	137				161	172	179				0	1.5	13	.030	—
No. of Samples	346	346	344	346	194	345	354	193	351	471				195	340	347				41	57	63				182	314	323	350	0
1914																														
Max. daily	F.M.	0	27.5	26.	90	30	0	130	118	48				222	220	214				235	232	233				8	8	39	.095	.028
Min. daily	0	0	0.5	1.	3	2	0	19	19	11				100	111	61				107	115	84				0	0	4	Tr.	.000
Aver. daily	0	0	10.6	11.	17	10	0	49	46	17				162	162	140				169	169	169				1.3	1.3	13	.033	.004
No. of Samples	365	365	364	362	322	364	720	322	364	365				322	364	364				51	63	69				322	364	364	732	339
1915																														
Max. daily	0	0	25	24.	55	20	0	86	79	28				207	217	210				218	229	230				6	9	30	.071	.022
Min. daily	0	0	1	1.5	2	2	0	22	20	9				120	123	69				127	130	103				0	0	24	.000	.000
Aver. daily	0	0	10	10.4	17	9	0	47	43	14				155	161	140				167	173	171				0	2	12	.017	.000
No. of Samples	365	365	365	365	373	365	1078	273	365	1076				273	365	365				43	69	70				274	365	368	718	241
1916																														
Max. daily	0	0	28	28	150	55	0	84	78	27				202	199	190				217	214	216				7	7	20	.025	.073
Min. daily	0	0	1	.1	1	1	0	21	21	9				77	80	52				82	85	79				0	0	4	.000	.000
Aver. daily	0	0	10	10	18	11	0	41	40	12				153	154	134				165	166	167				2	2	10	.002	.000
No. of Samples	366	366	366	366	359	366	1098	359	367	1100				359	366	366				61	61	61				359	366	367	701	283

NOTE: R.W. = River Water from force Main. S.B. = Sedimentation Basin. C.E. Combined Filter effluent. D.M. Filtered water from distribution main. M. = Marshy. F.M. Faint Marshy. T.R. Trace only.

Table 5—Summary of Sanitary Chemical Data (Parts Per Million)

	Mineral Residue Total		Mineral Residue Non Volatile		Mineral Residue Volatile		Chlorides			Nitrogen as free ammonia		Nitrogen as albuminoid ammonia		Nitrogen as nitrites		Nitrogen as nitrates		Oxygen Consumed		Dissolved Oxygen	
	S.B.	C.E.	S.B.	C.E.	S.B.	C.E.	R.W.	S.B.	C.E.	S.B.	C.E.	S.B.	C.E.	S.B.	C.E.	S.B.	C.E.	S.B.	C.E.	S.B.	C.E.
1913																					
Max.	246	251	144	145	102	106	2.8	—	2.9	.098	.060	.400	.296	Tr.	Tr.	.360	.257	21.1	9.1	—	—
Min.	189	194	97	109	82	71	0.9	—	1.6	.033	.019	.175	.115	Tr.	Tr.	.110	.075	3.5	2.7	—	—
Aver.	208	210	115	128	91	80	1.5	—	2.1	.058	.032	.292	.193	Tr.	Tr.	.223	.159	11.4	5.3	—	—
No. of samples	16	13	16	13	16	13	17	0	17	16	16	9	9	16	16	16	16	16	16	0	0
1914																					
Max	240	240	139	141	103	105	2.5	—	3.8	.115	.108	.353	.205	Tr.	Tr.	.292	.276	17.5	7.3	—	—
Min	187	191	99	115	82	71	1.5	—	2.5	.011	.015	.214	.152	.00	.00	.107	.083	4.9	3.8	—	—
Aver.	204	211	118	127	92	84	2.0	—	3.2	.043	.041	.270	.186	Tr.	.00	.192	.152	9.6	5.2	—	—
No. of samples	29	29	29	29	29	29	30	0	30	30	30	30	30	30	30	30	30	29	29	0	0
1915																					
Max.	241	241	141	141	100	100	3.0	3.0	4.0	.158	.133	.335	.226	.003	.002	.281	.251	12.0	4.9	10.6	11.2
Min.	180	.183	100	107	80	66	2.0	2.0	2.5	.006	.006	.139	.116	.000	.000	.070	.062	4.8	3.2	7.7	8.0
Aver.	210	212	121	132	89	80	2.4	2.4	3.3	.041	.035	.222	.143	Tr.	Tr.	.129	.109	7.9	4.3	9.6	9.8
No. of samples	20	20	20	20	20	20	19	19	19	19	19	19	19	17	17	18	18	19	19	15	15
1916																					
Max.	220	220	130	133	95	89	—	2.8	3.0	.244	.161	.308	.170	.004	.003	.252	.190	15.1	4.8	10.3	10.3
Min.	140	137	78	93	62	44	—	1.5	1.8	.007	.008	.135	.073	.000	.000	.036	.030	4.4	2.8	9.4	9.5
Aver.	192	192	110	119	82	73	—	1.8	2.1	.064	.044	.228	.137	Tr.	0	.118	.091	8.3	3.8	9.8	9.9
No. of samples	12	12	12	12	12	12	0	12	12	12	12	12	12	12	12	12	12	12	12	12	12

NOTE R.W. = River Water from force main. S.B. = Sedimentation basin. C.E. = Combined filter effluent. Tr. = Trace only.

Table 6—Summary of Analysis of Mineral Residues (Parts Per Million)

	Sedimentation Basin											Combined Filter Effluent										
	K+Na Potass +Sodium	NH_4 Ammon	Mg Magnes	Ca Calc.	Fe Iron	Al Alu.	HCO_3 Bicarb	NO_3 Nitrate	Cl Chloride	SO_4 Sulphate	SiO_3 Silica	K+Na Potass +Sodium	NH_4 Ammon	Mg Magnes	Ca Calc.	Fe Iron	Al Alu.	HCO_3 Bicarb	NO_3 Nitrate	Cl Chloride	SO_4 Sulphate	SiO_2 Silica
1913					$Fe_2O_3+Al_2O_3$											$Fe_2O_3+Al_2O_3$						
Max.	5.9	0.11	18.6	55.7	12.6		266	1.59	2.0	8.4	18.5	6.3	0.08	18.3	55.7	13.4		260	1.14	2.8	56.5	17.6
Min.	1.9	0.04	6.1	34.5	1.7		166	0.49	1.0	6.6	11.5	2.8	0.02	2.8	35.3	1.5		114	0.33	1.6	37.5	8.5
Aver.	3.0	0.07	11.7	39.4	5.2		189	0.88	1.4	7.4	14.5	3.9	0.05	9.6	40.0	6.6		155	0.44	2.0	40.4	10.9
No. of samples	10	10	10	10	10		10	10	10	10	10	6	6	6	6	6		6	6	6	6	6
1914																						
Max.	7.7	0.15	17.1	52.7	0.99	3.64	249	1.29	2.5	9.2	17.6	7.4	0.14	16.7	52.3	0.29	4.13	249	1.22	3.5	53.9	13.5
Min.	3.4	0.01	9.9	34.3	0.14	0.46	146	0.47	1.5	4.7	10.1	3.1	0.02	8.5	34.5	0.01	0.62	78	0.37	2.7	10.7	5.8
Aver	5.2	0.06	13.7	42.4	0.45	1.16	200	0.84	2.0	6.6	13.6	5.0	0.06	13.1	42.1	0.09	1.40	174	0.73	3.2	27.5	9.7
No. of samples	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12
1915																						
Max	7.7	0.20	17.1	51.4	0.58	1.01	247	1.28	3.0	10.7	15.2	7.6	0.17	17.1	51.8	0.06	1.6	235	1.16	4.0	50.6	12.2
Min.	5.1	0.01	10.7	34.7	0.18	0.13	157	0.31	2.0	4.6	9.8	5.2	0.01	10.2	35.2	0.01	0.4	104	0.26	2.5	13.8	6.3
Aver.	6.4	0.05	13.9	42.0	0.35	0.51	198	0.59	2.4	8.0	11.7	6.5	0.05	13.8	42.0	0.05	0.8	173	0.47	3.2	28.4	8.5
No. of samples	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12
1916																						
Max.	9.3	0.31	16.0	51.7	0.66	1.03	231	1.14	2.8	8.7	16.8	9.6	0.26	16.0	51.1	0.07	1.12	219	0.86	3.1	45.4	11.9
Min.	5.5	0.01	8.3	25.7	0.11	0.24	116	0.16	1.5	5.1	8.6	5.4	0.01	8.2	26.1	0.03	0.33	85	0.13	1.8	13.6	5.9
Aver.	7.4	0.09	12.8	40.1	0.33	0.46	189	0.46	1.9	6.5	12.8	7.3	0.06	12.7	39.9	0.05	0.63	164	0.36	2.2	26.7	8.9
No. of samples	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12

tered during any one month was in July, 1916, with 1,203,628,000 gallons and the maximum amount filtered during a twenty-four-hour day was on July 28th, 1916, when 58,929,000 gallons were put through the filters.

The average number of filters in service during 1916 was 15, as compared with 13 in 1915. The rate of filtration was reduced to an average of 2.1 million gallons per day in 1916 for each filter, or in other words, to a rate of 80 million gallons per acre of sand surface per day. This would be 64 per cent. of the normal rate of filtration for which the filters were constructed, namely, 125 million gallons per acre per day. The additional filters in service during 1916 also caused an increase of 20 per cent. in the length of filter runs and a decrease of 10 per cent. in the amount of wash water used for 1915.

Coagulant.—The amount of alum used in 1916, while greater in total amount because of the greater amount of water filtered, was 3.4 per cent. less than the amount used in 1915, if we consider the number of grains of alum per gallon of water treated. The cost of alum per million gallons of water for each of the four years was as follows: 1913, $4.66; 1914, $4.11; 1915, $3.47, and 1916, $4.25. The increase in cost for alum in 1916 was due to the European war, the price of

Table 7—Bacteriological Data

	Colony Counts on Agar at 37° and Gelatine at 20°C.															
	Raw Water		Settling Basin		Coag. Basin		Infl. to filters		Effl. from filters		Effl. from plant		Distribution Main		City Taps	
	Agar	Gel.	Agar	Gel.	Agar	Gel.	Agar	Gel.	Agar	Gel.	Agar	Gel.	Agar	Gel.	Agar	Gel.
1913																
Max.	2300	10,000	1200	5400	900		650				130	150	11	210	800	4500
Min.	70	125	5	120	3		2				0	1	1	3	0	1
Aver.	645	1565	240	395	155		85				7	11	4	24	80	215
No. of Samples	196	194	599	381	599		337				599	380	36	35	341	339
1914																
Max.	2700	5300	1000	8800	1500		1700		2200		155	160	170	13,500	150	11,000
Min.	55	250	40	150	20		4		3		0	1	0	1	2	2
Aver.	510	1000	255	790	155		120		195		12	11	14	200	16	310
No. of Samples	322	322	730	365	728		586		542		730	365	337	337	87	83
1915																
Max.	2600	12,700	1700	10400	1100		1400		2100		210	160	160	1100	215	400
Min.	40	120	38	110	14		10		2		0	1	1	1	1	1
Aver	1710	1720	335	860	152		195		70		8	8	7	24	14	38
No. of Samples	274	274	729	364	729		720		744		728	364	241	241	97	97
1916																
Max.	33,500	42,000	7300	52,000	2200		2000		800	65	800	2000	50	475	500	475
Min.	30	120	30	110	20		10		1	10	1	1	1	1	1	2
Aver.	775	2250	390	1345	160		165		90	35	13	27	7	24	18	25
No. of Samples	359	358	732	364	731		696		699	17	840	727	253	252	142	142

A. Aftergrowths in clear water reservoir

Table 8—Bacteriological Data

	B. Coli Determination (Percentage of total number of tests positive)																					
	River Water				Settling Basin				Coag. Basins				Influent to filters				Effl. from filters			C.E.	D.M.	City Taps
	10 C.C.	1 C.C.	1/10 C.C.	1/100 C.C.	10 C.C.	1 C.C.	1/10 C.C.	1/100 C.C.	10 C.C.	1 C.C.	1/10 C.C.	1/100 C.C.	10 C.C.	1 C.C.	1/10 C.C.	1/100 C.C.	10 C.C.	1 C.C.	1/10 C.C.	10 C.C.	10 C.C.	10 C.C.
1913																						
Max. Monthly	100	96			100	83			98	72			69	30						20	0	17
Min. Monthly	93	40			48	3			72	0			2	0						0	0	0
Aver.	99	74			86	38			66	23			36	13						2	0	5
No. of Samples	198	198			595	595			595	595			337	250						597	36	341
1914																						
Max. Monthly	100	100	84	44	100	100	92	81	100	97	74	33	98	92	86	75			64	3	4	50
Min. Monthly	100	86	12	0	96	73	8	0	93	30	3	0	47	10	4	2			13	0	0	0
Aver	100	97	50	10	98	90	45	22	97	68	33	9	60	39	46	17			39	1	1	13
No. of Samples	79	317	263	211	178	728	612	429	299	727	611	428	307	534	330	279			542	728	349	87
1915																						
Max. Monthly		100	77	13		100	62	21		93	47	11		90	36	11		72	21	7	3	60
Min. Monthly		44	6	0		42	10	0		3	2	0		3	2	0		3	0	0	0	0
Aver.		92	49	6		79	34	8		55	19	4		55	18	4		33	.7	4	0	12
No. of Samples		274	274	274		729	729	729		729	729	729		720	720	720		744	744	728	224	98
1916																						
Max. Monthly		100	67	37		98	76	38		85	32	19		83	45	12	95	52	0	37	10	43
Min. Monthly		75	9	0		46	11	0		20	2	0		19	3	0	19	0	0	0	0	0
Aver.		89	42	10		78	29	9		47	11	3		45	15	3	65	16	0	11	2	6
No. of Samples		359	359	359		731	731	731		731	731	731		727	727	727	730	730	62	861	252	144

Notes: City tap samples largely from "dead ends". C.E. = Filter Plant Effluent. D.M. = Distribution Main.

Table 9—Summary of Microscopical Examinations

	Diatomaceae											Chlorophyceae						Cyanophyceae		Protozoa		Rotifers	Miscellaneous	Amorphous Matter	Total	
	Navicula	Cyclotella	Nitzschia	Encyonema	Cocconema	Asterionella	Synedra	Cocconeis	Stephanodiscus	Melosira	Fragilaria	Scenedesmus	Protococcus	Conferva	Raphidium	Spirogyra	Pediastrum	Oscillaria	Anabaena	Dinobryon	Euglena	Anuraea				
	No. per C.C.	No. per C.C.	No. per C.C.	No. per C.C.	No. per C.C.	No. per C.C.	No. per C.C.	No. per C.C.	No. per C.C.	No. per C.C.	No. per C.C.	No. per C.C.	No. per C.C.	No. per C.C.	No. per C.C.	No. per C.C.	No. per C.C.	No. per C.C.	No. per C.C.	No. per C.C.	No. per C.C.	No. per C.C.	No. per C.C.	Stand. Units	No. per C.C.	Stand. Units
1913																										
Max.	214	1285	473	56	42	82	18	46	14	8	2	50	38	22	36	12	4	5		15		Pr.	38	340		
Min.	120	40	0	12	8	0	0	2	0	0	0	0	0	0	0	0	0	0		0			0	170		
Aver.	176	691	240	39	28	32	10	15	4	4		13	19	3	10	2	1	2		10			12	265	1297	640
Present in samples	10	10	9	10	10	8	10	10	9	6	3	8	8	4	8	2	7	4	0	8	0	1	8	10	10	
1914																										
Max.	138	2479	459	58	39	85	12	24	15	31	5	35	18	36	18	14	5	12	2	31	1	2	26	520		
Min.	40	40	6	4	7	0	0	0	.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	100		
Aver.	104	695	101	23	20	16	6	8	6	8	2	14	6	9	7	3	1	4	1	9	1	1	10	290	1070	402
Present in samples	35	35	35	34	35	33	34	34	34	33	19	33	33	24	31	20	10	27	10	29	13	12	33	35	35	
1915																										
Max.	238	2480	386	62	30	125	9	23	26	26	1	29	7	10	8	4	2	4	1	22	1	2	27	388		
Min.	69	48	5	7	6	0	0	4	1	3	0	1	0	0	0	0	0	0	0	0	0	0	3	98		
Aver.	127	317	65	21	18	20	4	11	10	11	1	11	2	4	3	1	1	1		7		1	9	237	506	320
Present in samples	43	43	43	43	43	34	40	43	43	43	25	43	31	39	26	29	24	8	4	33	4	14	43	43	43	
1916																										
Max.	116	490	174	9	23	55	9	17	37	37	4	19	10	9	8	4	6		4	11		2	32	875		
Min.	57	21	2	2	5	0	0	4	0	2	0	1	0	0	0	0	0		0	0		0	3	70		
Aver.	86	156	41	5	11	9	3	10	10	16	1	8	2	4	4	2	2		1	3		1	11	271	374	266
Present in samples	39	39	39	39	39	27	36	39	36	39	12	39	19	33	33	23	23	0	6	26	0	14	39	39	39	

Table 10—Typhoid Fever Statistics

	Number of Typhoid Cases	Number of Typhoid Deaths	Death rate per 100,000 pop.	Remarks.
1900	376	79	38	
1901	630	121	58	
1902	320	66	29	
1903	720	95	39	
1904	738	103	41	
1905	269	62	23	
1906	252	97	34	
1907	181	77	26	
1908	104	51	17	
1909	95	59	19	
1910	1252	173	57	Chlorination begun Feb. 1910
1911	299	36	11	
1912	186	37	11	
1913	136	41	12	Filter Plant in operation Jan. 10, '13
1914	278	38	12	Pop. 343,460
1915	166	25	7	Pop. 353,460
1916	181	21	5.8	Pop. 363,460

alum having advanced approximately 37 per cent. A large stock of alum carried over from 1915 helped to keep down the alum cost for 1916.

Sterilization.—Hypochlorite of lime had been used for sterilization from the time the filtration plant was put in operation until December 6th, 1915. At that time it was replaced by liquid chlorine, due to the scarcity and high price of hypochlorite, caused by the war.

The cost for sterilization in 1916 was $0.27 per million gallons, as compared with $0.21 in 1915. If hypochlorite had been used in 1916 the cost would have been fully 1,000 per cent. greater for sterilization, the price of hypochlorite having advanced from $0.015 to $0.20 maximum per pound in 1916. A stock of 5,640 pounds of hypochlorite of lime which was left over when the liquid chlorine treatment begun was sold by the purchasing department at an average price of $0.112 per pound, a net gain to the city of approximately $0.09 per pound.

There have been no complaints of odors or tastes in the filtered water due to the sterilizing agent since the liquid chlorine treatment was installed. The Wallace & Tiernan chlorine machines have given very satisfactory service. Table 2.

Total Cost of Operation.—The total cost per million gallons for purification of the water was $9.18 in 1913; $9.40 in 1914; $8.38 in 1915, and $8.90 in 1916. The cost for 1916 shows an increase of 6.2 per cent. over that for 1915, the chief increase in unit costs being for coagulant and sterilizing agent, which advanced in price in 1916. There were decreases in several of the other unit costs for 1916 as compared with 1915.

Laboratory Data.—The average turbidity of the river water as it is received in the sedimentation basin at the filtration plant has remained practically the same for each of the four years that the plant has been in operation, the maximum of 150 parts per million occurring in 1916.

The average yearly color of the river water has decreased steadily from year to year as may be seen in Table 11. The reason may possibly be found in the decrease in the logging industry along the Mississippi River above Minneapolis, and also in the increasing demand for agricultural lands with the consequent draining of swampy areas. A maximum color of 130 parts per million occurred in the river water in 1914, as compared with 86 parts in 1915, and 84 parts in 1916. The average color of the filtered water was 18 parts per million in 1913; 17 parts in 1914; 14 parts in 1915, and 12 parts in 1916.

The average alkalinity of the river water has remained practically the same from year to year. It varies during the year from a maximum of 224 parts per million to a minimum of 80 parts.

The average total hardness also has remained practically the same. It varies from 248 parts per million to

Table 11—Comparison of Color in River Water at Minneapolis with Average Monthly Precipitation on Mississippi River Watershed Above Minneapolis

	Jan.	Feb.	Mar.	April	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Summary
1913													
Average Precipitation	0.32	0.36	1.11	2.00	3.07	3.03	7.18	1.57	3.63	3.12	0.48	0.02	25.89
Color Max.	17	17	38	41	51	59	108	93	71	61	50	42	108
Color Min.	11	14	17	28	29	50	48	55	48	49	38	38	11
Color Aver.	14	16	25	33	43	55	76	74	58	63	45	40	47
1914													
Average Precipitation	0.74	0.43	1.14	2.45	2.43	8.36	2.56	5.51	3.67	1.81	0.41	0.18	29.71
Color Max.	38	23	26	42	81	1.30	124	75	60	55	40	30	130
Color Min.	23	19	20	37	42	58	75	40	32	38	31	25	19
Color Aver.	29	20	22	39	64	99	91	53	42	45	34	27	49
1915													
Average Precipitation	0.67	1.20	0.42	1.73	4.00	8.54	3.64	1.53	2.87	2.13	1.89	0.73	29.35
Color Max.	25	23	72	42	77	75	86	55	29	47	79	60	86
Color Min.	22	20	27	30	40	46	57	32	24	22	35	30	20
Color Aver.	23	21	38	36	53	63	75	43	27	37	54	42	44
1916													
Average Precipitation	2.05	0.38	1.63	2.06	4.94	5.14	2.47	5.23	3.89	1.42	0.16	0.79	29.57
Color Max.	30	25	50	75	84	72	82	50	46	37	28	26	84
Color Min.	24	21	24	49	68	51	39	30	33	27	25	21	21
Color Aver.	27	23	29	64	75	60	60	35	39	31	26	23	41

Note: Precipitation data from U.S. Weather Bureau for stations at Bemidji, Cass Lake, Brainard, St Cloud and Minneapolis

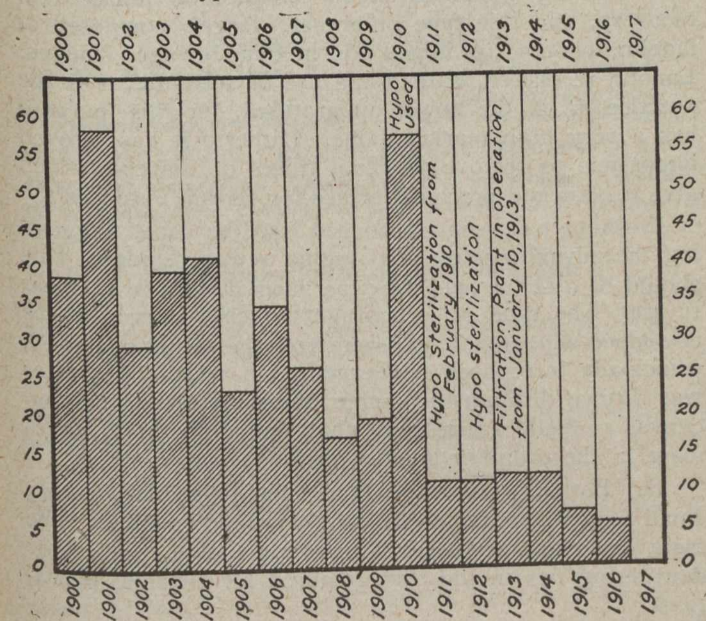
Table 12—Relation of Amount of Water Filtered (Million Gallons) to Precipitation and Temperature Data at Minneapolis

	Jan.	Feb.	Mar	Apr.	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Summary
Normal Precipitation	0.69	0.76	1.65	2.44	3.92	4.01	3.81	3.69	3.66	2.58	1.18	0.95	29.34
Normal Temperature	13.7	15.1	29.4	46.6	57.3	67.2	72.1	69.6	62.0	49.7	33.0	20.1	44.6
1913													
Precip.	0.42	0.74	1.65	1.86	2.86	2.21	7.75	1.40	4.12	2.55	0.48	0.05	26.09
Temp. F.	13.0	12.7	25.7	49.0	55.8	70.5	70.4	72.8	61.4	46.4	40.2	30.9	45.7
Water Filtered Total	568.585	674.858	679.159	673.199	703.525	1072.735	887.488	1007.002	829.899	786.337	681.982	649.429	9.294.198
Water Filtered Daily Aver.	23.691	24.102	21.908	22.440	25.275	35.758	28.626	32.485	27.663	25.366	22.733	20.949	26.218
Water Filtered Max. Daily	25.821	26.425	25.916	25.146	30.473	45.817	39.069	41.768	37.506	31.430	24.770	29.266	45.817
1914													
Precip.	0.83	0.45	0.98	3.69	1.80	8.63	1.17	8.70	2.76	1.58	0.19	0.37	31.15
Temp. F.	21.4	7.6	30.6	44.7	60.4	66.8	75.1	69.2	62.4	55.4	36.0	12.2	45.2
Water Filtered Total	656.574	610.644	705.849	678.582	816.267	772.499	1.052.907	879.375	769.077	763.172	721.946	754.948	9.175.840
Water Filtered Daily Aver.	21.148	21.809	22.770	22.620	26.331	25.750	33.965	28.173	25.636	24.619	24.065	24.353	25.413
Water Filtered Max. Daily	23.290	26.033	26.706	26.824	33.059	34.773	44.949	38.557	29.622	30.138	26.770	28.759	44.949
1915													
Precip.	1.37	2.01	0.93	1.87	3.98	4.91	6.92	3.49	2.57	2.59	3.62	0.46	33.72
Temp. F.	12.6	25.4	27.8	56.0	52.2	62.5	67.2	65.4	60.6	51.4	35.4	22.6	44.9
Water Filtered Total	761.226	658.648	720.465	739.168	780.158	790.606	843.216	875.435	871.765	813.526	750.997	770.471	9.375.681
Water Filtered Daily Aver.	24.556	23.523	23.242	24.639	25.166	26.353	27.200	28.240	29.059	26.243	25.033	24.854	25.687
Water Filtered Max. Daily	27.443	26.314	26.047	30.659	32.908	33.299	33.038	32.017	36.806	30.437	27.647	28.802	36.806
1916													
Precip.	2.88	0.32	1.19	3.07	6.97	4.54	1.27	1.66	2.42	1.60	0.58	0.98	27.48
Temp. F.	10.0	11.0	26.0	43.6	56.9	62.9	79.0	72.0	59.8	46.9	34.6	12.4	42.9
Water Filtered Total	791.914	739.577	791.361	791.147	864.922	831.770	1.203.628	1.140.195	941.466	899.210	836.735	888.095	10.719.969
Water Filtered Daily Aver.	26.546	25.503	25.528	26.372	27.901	27.726	38.827	36.779	31.382	29.007	27.891	28.648	29.290
Water Filtered Max. Daily	28.570	28.460	28.728	27.970	35.880	33.202	68.929	52.348	40.269	34.014	32.968	33.322	68.929

82 parts during the year and averages 165 parts per million for 1916.

Free carbonic acid in the filtered water showed an average of 10 parts per million in 1916 as compared with 12 parts in 1915 and 13 parts in 1914 and 1913. Free carbonic acid in the filtered water is due chiefly to decomposition of the bicarbonates of calcium and magnesium by the aluminum sulphate added and is chiefly of interest because of its corrosive action on the iron pipes in hot water systems thereby causing "red water."

Chart I.—Typhoid Fever Death Rate Per 100,000



PRESSING OF SEWAGE SLUDGE*

F. A Dallyn, C.E.,
Ontario Provincial Sanitary Engineer

SEWAGE sludge is now recognized as having a value, mainly for its nitrogen content and for the mineral salts present, together with the general improvement in humus following its introduction. The results of experiments, especially those in which its usefulness was condemned and reported in the transactions of the Royal Commission on Sewage Disposal, should not discourage us in this country from further effort. The soil conditions are materially different in this country, as well as the methods of agriculture. With us the great cost of transportation of prepared fertilizers should tend to encourage the use of sewage sludge locally.

If the sludge is to be dried to the 10 per cent. basis now commonly required for fertilizer compounds, or as a filler for fertilizer replacing the abattoir tankage, which is fast disappearing owing to the development of prepared stock foods, it will require drying in a temperature sufficient to sterilize all bacterial organisms. Where this has not been done the material in fine powder form has been shown to have an injurious effect upon workmen handling it, in some instances occasioning violent skin eruptions after its inhalation in large quantities.

The sludge containing over 3 per cent. nitrogen on a dry basis may be said to be marketable, and the difficulty is merely to determine the particular point where it can be used with greatest advantage. In England the export of sewage sludge has largely been to France, Germany and the States. The markets were fairly good. The American market, however, was shut off owing to the fact that sludge with a higher moisture content than 10 per cent. gave a good deal of trouble from heating during the voyage, and this, more than anything else, was responsible for the discontinuance of shipments. The grease recovery from sludge, introduced in this country by Mr. Dorr, supported by his experiments at Boston, has attracted a good deal of attention. Too little attention, however, seems to have been paid to the quality of grease and to the effects of extraction.

Grease in sewage divides, naturally, into two classes, mineral oils and fatty oils. In the latter the percentage of animal fats is somewhat higher than that of vegetable oils. It is unreasonable to expect to be able to discover grease in a sewage sludge which has not been introduced directly to the sewers, either by domestic wastes, abattoirs, manufacturing or wool-washing industries, and the success of municipal recovery will depend largely upon the fact that the manufacturers and abattoirs are dilatory in practising the art of local recovery. A local census of the industries dscharging to any given sewerage system and an inventory of the proportion of their material which reaches the sewers should be undertaken before attempting a report upon the advisability of grease recovery on a municipal scale at sewage disposal works. Generally speaking, there would not be a sufficient amount of grease in sewage sludge to make recovery a paying proposition or to interfere with its fertilize value if the manufacturers were alert and recovered grease locally.

The presence of considerable quantities of grease interferes very materially with the pressing operations, and the use of neither acid nor lime removes this handicap. The action of lime is to saponify the grease and to form an insoluble lime soap. Treatment of alkaline sewage with acid occasions a very similar reaction, the acid saponification yielding 55 to 63 per cent. fatty acid as against 45 to 47 per cent. by the lime process.

The advantage which has been observed in the Miles process lies not so much in the greater effectiveness of the acid process, but almost solely to the fact that with the acid the molecular weight of the fatty acid is increased, the tri-stearate being fromed instead of the stearate. The acid process in the soap is not generally used commercially except for the manufacture of fatty acids from inferior fats, such as those obtained from garbage, putrid bones, etc.

Mr. Allen appears to have overlooked a very important matter in connection with the pressing of sewage sludge, and that is that the soaps and greases present in sludge are frozen or solidified at the ordinary sewage temperatures, and in this condition are capable of clogging the filter cloths.

The following table gives the melting and solidification points of a few of the fatty acids from various oils and fats which may be encountered in sewage sludge, and you will note that temperatures in excess of 69° C. are necessary to melt this material:—

*Abstracted from discussion of paper by Kenneth Allen on same subject and printed in Transactions of the American Society of Municipal Improvements.

Solidification Points, Degrees Centigrade

Beef tallow	38-46	Palm oil	36-45
Mutton tallow	41-48	Cocoanut oil	20-25
Lard	34-42	Palm-nut oil	20-25
Horse fat	33.7	Japan wax	59
Neatsfoot oil	26.5	Vegetable tallow	45-53
Linseed oil	13.17	Olive	17-26
Tung oil	37	Arachis	23-29
Hemp oil	14-16	Cold-liver	18-24
Poppy-seed	16.5	Whale	24
Cotton-seed	32-36	Japanese sardine	28
Rape	12-18		

Melting Points, Degrees Centigrade

Oleic	4	Elaidic	51
Oxystearic	84-86	Stearic	69
Iso-oleic	44-45	Palmitic	62

This would readily account for the success of Mr. Garfield in his sludge-pressing and de-greasing work at Bradford. The acid treatment which is in use there, and which I had the pleasure of enquiring into a year ago, is introduced mainly to promote precipitation, the sewage being quite alkaline at the works. Precipitation starts when the sewage is about 8 grains alkaline to methly-orange. The cost of heating the presses is not so material as one might generally be led to believe, in view of the fact that no evaporation takes place, and it is only the specific heat and the temperature range which has to be taken into count. My recollection is that in Mr. Garfield's plant the total operation of the works required about one ton of coal to forty tons of wet sludge, that is, sludge containing about 82 per cent. moisture. It works out to about one ton of coal to a ton of sludge as it comes from the presses with about 25 per cent. to 28 per cent. moisture, and which is further reduced by the heat of the mass to about 15 per cent. or 16 per cent. moisture by simply piling in the storage yards. This further improvement is assisted by the fact that after coming from the presses the sludge is made into egg-shaped briquettes, as has been indicated by Mr. Allen, which leave plenty of voids for the circulation of air. The introduction of heat has a further benefit in the fact that the viscosity of the water and of the melted fats is very much improved, and they escape readily from smaller openings or with higher velocities from the same opening.

As to the market for grease recovered from sludge, as far as I have been able to determine, there is but one considerable market, and that is for the manufacture of textile soaps. Some years ago it would probably also have been introduced into lubricants, but with our present knowledge of the behavior of the fatty oils in the presence of oxygen, which tends to liberate free fatty acids which have a corrosion action, it would appear that for the future the mineral oils will be used exclusively for that purpose.

The mineral oils do not materially interfere with pressing operations. Lubricating oils and greases which escape to the sewers will not tend to clog the press cloths. They are mainly manufactured from mineral oils and seldom solidify above 15° C. in winter, 5° C. in summer, different classes of oil being thus permitted at different seasons. Their viscosity, however, may be markedly improved by increasing the temperature of the sludge.

It is worth noting that the glycerol content of grease extracted from sewage will be low in view of the fact that fats in the presence of nitrogenous animal or vegetable impurities decompose readily into free fatty acids and glycerol by enzyme hydrolysis. The acids will, in some of the processes, absorb oxygen and form the oxyacetic. The glycerol is also partially destroyed.

Grease extracted by solvent processes, which is about the only process left when the grease content is not in excess of 20 per cent., limits the market to its use in the soap industry or as adulterant to lubricating oils. The wool-washing industries should advance the art of local recovery, and this type of grease, largely composed of lanolin, cannot be hoped for in our sewage sludges. Lanolin recovered without the use of solvents, as is the practice in all the newer apparatuses, for this purpose has a very high market value. Hitherto it has proven impossible to remove the final traces of solvents which give it odor and preclude its use for certain purposes.

Nothing has been introduced into the paper suggesting the advantages of centrifuging sewage sludge. This should be a fertile field for experimental work in the near future. The type of centrifuge which I expect to see developed will be quite different from the earlier machines, principally introduced in Germany, in which the water was thrown through the sludge, the sludge being held on canvas or metal gauze somewhat similar to the arrangement in the ordinary laundry drier.

Dr. Bartow's experiments at Champaign with a very small centrifuge have shown very good results. The main disadvantage of centrifuges for sludge recovery or dewatering lies in the time required for cleaning operations.

In conclusion, it would seem practicable to group our incinerator and sludge-pressing plants and take some advantage of the heat usually available at such plants, for the English experiments have shown that there is no difficulty in conveying sludge long distances through pipes. The combination of the two will work just as well if centrifuges are introduced and low-pressure steam with vacuum condenser used for motive power.

It is the writer's confident opinion that the utilization of sewage sludge is not by any means a remote possibility, and that we may expect satisfactory returns from its manurial values as soon as we determine its limitations and make its application coincide more closely with soil requirements.

CLASS FOR FEMALE TRACERS

IN Detroit, Michigan, a class has been started at the Cass Technical High School to teach tracing to women.

There are no tuition fees and the necessary equipment is provided free of charge. Class hours are from 8 a.m. to 1 p.m. every day excepting Saturday and Sunday, and the course takes eight weeks. If there are a sufficient number of registrants, other classes will be conducted in the evenings. It is suggested by the school that women between the ages of eighteen and thirty are preferred, although no age limit has been actually established.

Condemnation of well waters as the result of inspections and analyses by the Illinois State Water Survey for the nine-year period from 1907 to 1916 shows that as the depth of the wells increases the percentage condemned as unsafe decreases. During the period named 11,281 wells were examined and of these 5,091, or 45 per cent., were condemned. For wells less than 25 ft. deep the percentage of condemnation was 74; 25 to 50 ft., 63; 50 to 100 ft., 36; more than 100 ft. deep, 14. Of wells with unknown depths the percentage of condemnation was 44, the total number of wells being 636.

The Canadian Engineer

Established 1893

A Weekly Paper for Canadian Civil Engineers and Contractors

Terms of Subscription, postpaid to any address:

One Year	Six Months	Three Months	Single Copies
$3.00	$1.75	$1.00	10c.

Published every Thursday by
The Monetary Times Printing Co. of Canada, Limited

JAMES J. SALMOND *President and General Manager*
ALBERT E. JENNINGS *Assistant General Manager*

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Principal Contents of this Issue

U.S. RAILROAD SITUATION

SOME surprise was caused by the recent action of the United States Railroad Administration in handing back to the owners the entire control of approximately 2,000 United States railroads. These were, of course, short-line roads, electric feeders and plant facility lines. There remain in the hands of the railroad administration about 200 short-line railroads. It is obvious, therefore, that it is the intention of the government to retain control of the great roads which are essential to the industries of the United States, and to relinquish those roads which serve particular industries or outlying communities. Many of the returned roads were built merely to serve single industrial plants such as lumber concerns, coal mines, etc. About 200 are electric feeders to the main-line roads and less than 400 are short-line railroads doing the general business of a common carrier between two or more localities.

This action of the railroad administration has no doubt been influenced by the results of the first year's operation of the roads by the government. It was not necessary to operate these 2,000 short roads in order to determine whether they were essential to the industries of the country. It was clearly obvious in the first place that this was not the case. Had the government operation of the roads been successful in the economic sense, there is no doubt that these lines would have been retained, together with the longer lines. Their return, therefore, is a frank admission that the government cannot operate the lines with the same efficiency as the owners themselves. Whether the lines are operated by the owners or by the government, the former are going to do all in their power to make them safe and profitable investments. The work of the government cannot assist them in this direction and is, therefore, unnecessary.

In retaining the large lines, however, the government maintains with consistency its original policy. These are the ones upon which every industry depends to a greater or smaller extent. It is, therefore, for political, not for economic motives, that these are operated by the government. In fact, it is generally admitted by all who examine the results, that the expenses of operation have increased more rapidly than would have been the case under private control. Not only was a general increase in rates of 15 per cent. insufficient to meet this increase, but also the government found itself with a substantial deficit to meet. After many years of consistent restrictions of rates and consequent depreciation in the value of railroad securities, the United States government found itself almost immediately after assuming the business of railroading, required to make a much larger increase in rates than had been even considered while the railroads were in private hands.

ADOPTION OF THE METRIC SYSTEM OPPOSED IN GREAT BRITAIN

THE final report to the British Parliament of the Committee on Commercial and Industrial Policy after the War contains a chapter relating to weights and measures. The 19 members of the committee were unanimous in opposing the compulsory adoption of the metric system. Their conclusions are as follow:

"Having given very full consideration to the subject, we are unable to recommend the compulsory adoption of the metric system in this country. In our opinion, it is absolutely certain that the anticipated uniformity could not be obtained for a very long period, if ever.

"There is, further, the serious objection that if we induced the above mentioned countries to change over to the metric system, we should be surrendering to Germany the advantage which our manufacturers now enjoy over hers, both in their markets and our own.

"We are informed that even in France, which has made the metric system nominally compulsory for more than half a century, the 'pouce' (or inch) is used in textile manufacture and numerous local measures still survive.

"In referring to these considerations, we have to point out that there is no unanimity even as to the theoretical merits of the metric system as compared with our own. The practical argument that its adoption is desirable in order to secure uniformity in the markets of the world has been shown to be unfounded. We are not satisfied by any evidence which has been brought before us that trade has actually been lost to this country owing to the fact that the use of the metric system is not compulsory.

"But to attempt to make the use of the system universal and obligatory in this country would cause loss and confusion at a particularly inopportune moment, for the sake of distant and doubtful advantages. We are convinced that, so far from assisting in the re-establishment of British trade after the war, such a measure would seriously hamper it.

"As regards the educational advantages claimed for the change, we have been referred to a statement quoted by the Select Committee of 1895 that no less than one year's school time would be saved if the metric system were taught in the place of that now in use. The information which we have received does not support that statement, and even if it were well founded, it must be remembered that for at least a generation, children would have to learn both the new and the old measures and how to convert from one to the other.

"It is often popularly supposed that the introduction of the metric system would render possible the immediate sweeping away of many complicated and varying weights and measures. As we have already indicated, this belief is, in our opinion, wholly fallacious.

"We are not convinced that the metric system is, upon the whole, even theoretically superior to the British system, and we are satisfied that the practical objections to the proposed change are such as decisively to outweigh any advantages which are claimed for it."

PERSONALS

A. B. HUNTER recently assumed the duties of manager for the Federal Coals, Limited, at Lethbridge, Alta.

GEO. H. MADGETT, formerly with the Hamilton Bridge Works Co., is now connected with the Standard Steel Construction Co., of Welland, Ont.

G. L. FRASER has been appointed general manager of the Granby Consolidated M. S. & P. Company's coal mines, with headquarters at Nanaimo, B.C.

Lieut.-Col. GEORGE ERIC MCCUAIG, of Montreal, has been made a Companion of St. Michael and St. George. His name appeared in the list issued on the King's birthday.

W. H. BREITHAUPT, of Kitchener, Ont., has been re-elected chairman of the Kitchener City Planning Commission. N. HELLMUTH has been appointed secretary of the commission.

C. S. PARSONS, of the Mines Branch, Ottawa, has resigned to accept a position as superintendent of the Joseph Dixon Crucible Company's graphite mill and mine at Graphite, N.Y.

W. A. BUCHANAN, Bank of Hamilton Building, Toronto, has been appointed sales manager of the Canadian Incinerator Co., Limited, for Ontario, Quebec and the Maritime Provinces.

L. H. FORTIER, formerly superintendent of streets at Moose Jaw, Sask., and latterly of the staff of the Imperial Munitions Board at Toronto, has joined the Toronto sales department of the Canadian Fairbanks-Morse Co., Limited.

EDWARD J. ALBERT, for the past five years manager of the mining and power department of the Canadian Allis-Chalmers, Limited, has resigned to accept a position as sales manager of the Thwing Instrument Company, Philadelphia, Pa.

Capt. ERIC E. RYERSON, a graduate of the School of Applied Science, University of Toronto, 1909, and who prior to going overseas was a member of the firm of Mitchell & Ryerson, Toronto, has been mentioned in recent despatches of Gen. Sir Douglas Haig.

Lieut.-Col. J. L. R. PARSONS, formerly of Georgetown, Ont., who has recently been mentioned in Sir Douglas Haig's despatches, is a graduate of the School of Applied Science, University of Toronto, class of 1897. He enlisted in 1917 and won his promotion on the field.

H. R. SAFFORD, chief engineer Grand Trunk Railway System, announces the appointment of C. H. Tillet as electrical engineer of the system, with headquarters at Montreal. Mr. Tillet succeeds J. A. BURNETT, who has been appointed technical assistant with the British War Mission at Washington, D.C.

Lieut. ARTHUR C. OXLEY, recently mentioned in Sir Douglas Haig's despatches, went to France with the 2nd Field Co., first contingent. He received his promotion on the field and was given the D.C.M. for conspicuous bravery under fire. He graduated in civil engineering from the University of Toronto in 1910 and at time of enlisting was employed by the Dominion Bridge Co.

HARRY H. ANGUS, of MacMullen, Riley & Angus, consulting engineers, Toronto, has gone to New York City where he will stay for five or six months to assist the New York office of his firm in connection with war contracts. Mr. Angus will proceed to Camp Eustis, Virginia, at an early date to supervise the construction of a power plant and of a heating plant for hospital and other buildings.

Engineer-Lieut.-Commander HOWLEY, of the Canadian Naval College, is a veteran of the Dardanelles. Commander Howley is a native of St. Johns, Newfoundland, but has spent the greater part of his life in England where as a boy he joined the Naval Engineering College, spending twenty years at sea in the engineering branch of the navy. He has recently removed to his new home at Victoria, B.C.

OBITUARIES

ISAAC SHONE, well known as the inventor of the Shone pneumatic sewage ejector, died recently. At the time of his death Mr. Shone was eighty-two years old, having been born at Brymbo, England, in the year 1836.

DR. EDGAR MARBURG, professor in charge of the department of civil engineering of the University of Pennsylvania, Philadelphia, died on June 27th. Dr. Marburg was secretary-treasurer of the American Society for Testing Materials, having held that position for sixteen years. He exercised a powerful influence on the character, standing and usefulness of the society, and on the making of specifications and tests for materials of engineering.

EASTERN ONTARIO GOOD ROADS ASSOCIATION

THE recently organized Eastern Ontario Good Roads Association, of which Alderman Wm. Findlay, of Ottawa, is president, is conducting an aggressive campaign in an effort to improve the highways of Eastern Ontario.

The organization meeting of the new association was held in Ottawa July 8th and seldom has there been such a large and representative gathering of citizens from all parts of Eastern Ontario. Enthusiasm ran high and should the objective which the association has set before it be attained it will mean a great deal for the good roads movement generally and for the highways of Eastern Ontario specifically.

One hundred and sixty-six members were secured at the first meeting.

The officers of the new association are as follows:—President, Ald. William Findlay, Ottawa; 1st vice-president, John Brennan, Arnprior; 2nd vice-president, Dr. T. W. Smith, Hawkesbury; 3rd vice-president, W. F. Barker, Portland; directors, C. J. Foy, Perth; T. H. Moffat, Pembroke; W. F. Bell, Britannia; J. F. Ault, Winchester; D. A. McNaughton, Finch; D. E. Johnson, Ottawa; secretary-treasurer, W. Y. Dennison, Ottawa. The executive committee consists of the president, three vice-presidents, the secretary-treasurer, C. J. Foy and W. J. Fisher, Winchester.