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THE COQUITLAM-BUNTZEN HYDRO-ELECTRIC DEVELOPMENT

HARNESSING THE WATER POWERS OF COQUITLAM AND BUNTZEN LAKES, BRITISH COLUMBIA—NEW HYDRAULIC-FILL TYPE OF DAM AT COQUITLAM—WESTMINSTER WATER SUPPLY—POWER EXTENSIONS AT LAKE BUNTZEN

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THE Coquitlam Lake water power development is owned and operated by the Vancouver Power Company, Limited, which is a subsidiary company of the British Columbia Electric Railway Company, Limited, a company operating a railway, light and power business in the districts surrounding the cities of Vancouver and New Westminster, and the Fraser Valley, and also the city of Victoria and district on Vancouver Island.

The main features of the original power development, which were put into operation in December, 1904, consisted of a scheme connecting Lake Coquitlam with a lake known as Lake Buntzen by means of a tunnel through a mountain range 4,000 ft. high separating the two lakes, which has an elevation of about 400 ft. above the North Arm of Burrard Inlet.

At the outlet of Lake Buntzen a concrete dam was built 54 ft. high and 361 ft. long. From this dam originally four penstock lines were carried down 1,800 ft. to the power house, which is situated on a rocky bluff at the edge of the tidal waters of the North Arm. The original plant consisted of four 3,000 h.p. Pelton wheels, having an effective head of about 395 ft., and four 1,500 kw. Westinghouse generators.

Fig. 1.—General View of Coquitlam Dam During Construction.

Watershed.—The available water supply is derived from the Coquitlam watershed, which has an area of 105 sq. mi. The upper part of the watershed rises to 8,000 ft., and is covered with perpetual snow. Over this watershed the annual precipitation for the last eleven years has averaged 155 in. The maximum year during that period was in 1906, when the precipitation was 190 in. The minimum year was in 1911, with a record of 132 in. The maximum monthly precipitation is during the month of November. The average precipitation for November for the past ten years amounts to 28 in.; the maximum recorded rainfall during that period for the month amounts to 37¼ in.; the minimum recorded is 10.62 in. The minimum monthly rainfall is in July, the average for the past ten years being 2½ in. In the original project, Lake Coquitlam, which has an area of about 2,000 acres, was raised 11 ft., or from El. 432 ft. above sea level to El. 443 ft. The lake was raised by means of a rock-filled timber crib dam 169½ ft. in length, which was designed as an overflow dam, having a spillway 113 ft. long. The area of the lake formed by the old dam was 2,328 acres, and with the growth of the demand for electric energy, it was decided that practically all of the water running off from the watershed could be conserved if a dam were built at the outlet of the lake sufficiently high to store an additional 60 ft. of water, or a total depth of about 71 ft. that might be drawn down. Careful observation of the run-off data showed that slightly in excess of 1,000 cu. ft. per sec. was the average run-off for the whole year. This run-off represented about 78 to 80% of the recorded precipitation at Coquitlam Lake, and represents an average water storage of electric energy of 220,000,000

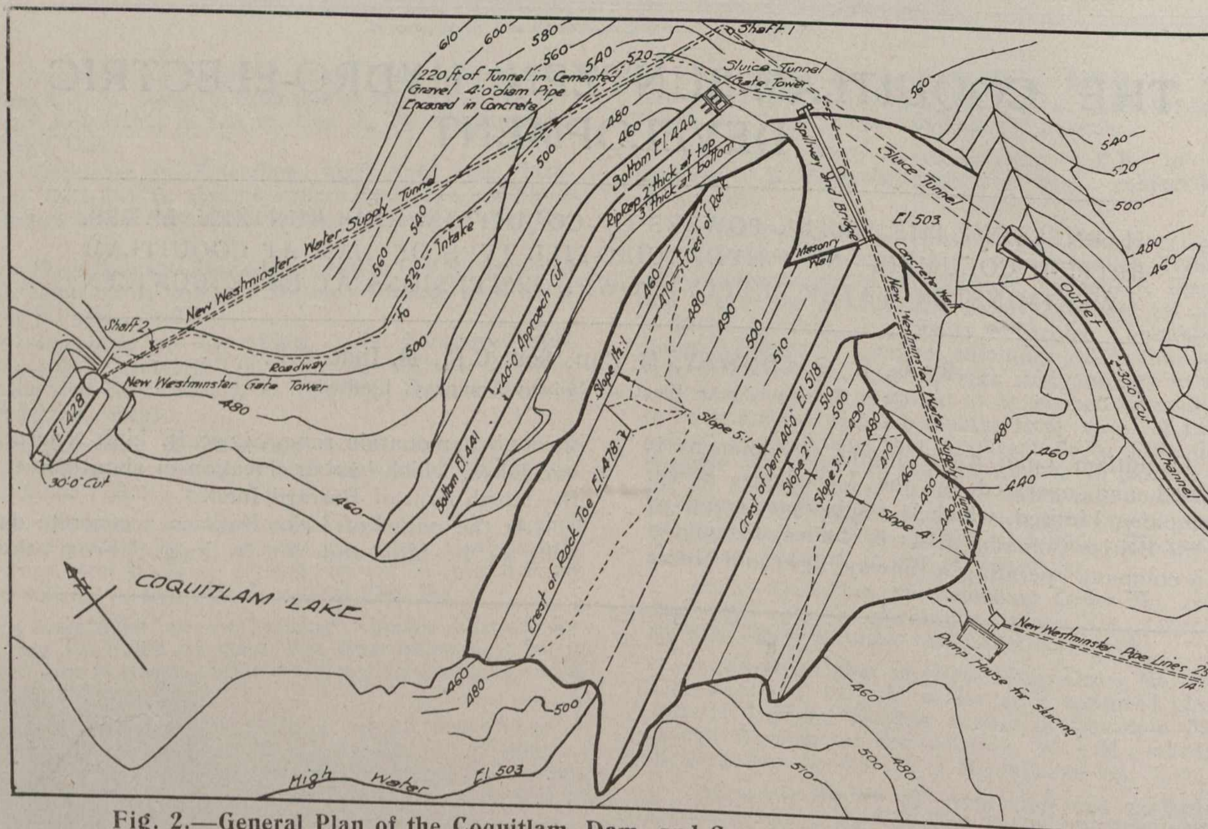


Fig. 2.—General Plan of the Coquitlam Dam, and Surrounding Development.

kilowatt-hours. To deal with this increased quantity of water it was necessary to enlarge the Coquitlam-Buntzen power tunnel, which originally had a cross-section of 81 sq. ft. and a capacity of 300 cu. ft. per sec., to 178 sq. ft. This work was completed in the spring of 1911, and the tunnel has now, when the lake is full, a maximum discharging capacity of 1,350 cu. ft. per sec. In addition to the water for Coquitlam Lake, Lake Buntzen, which has an area of 500 acres and a drainage of 7 sq. mi., has a considerable run-off. The rainfall at that lake averages 113 in. over the last ten years; the maximum year was 1906, when it was 137½ in., the minimum year 98½ in. 1911. The storage capacity of Lake Buntzen is 6,000 acre-feet.

The necessity of utilizing all the available water supply in Coquitlam Lake was due to the extraordinary growth which has taken place in the development of the City of Vancouver and surrounding districts during the last five years. The following figures represent the output of electric energy from the company's plants since 1908:—

Fig. 4.—Foundation Ready for Hydraulic Filling, Coquitlam Dam.

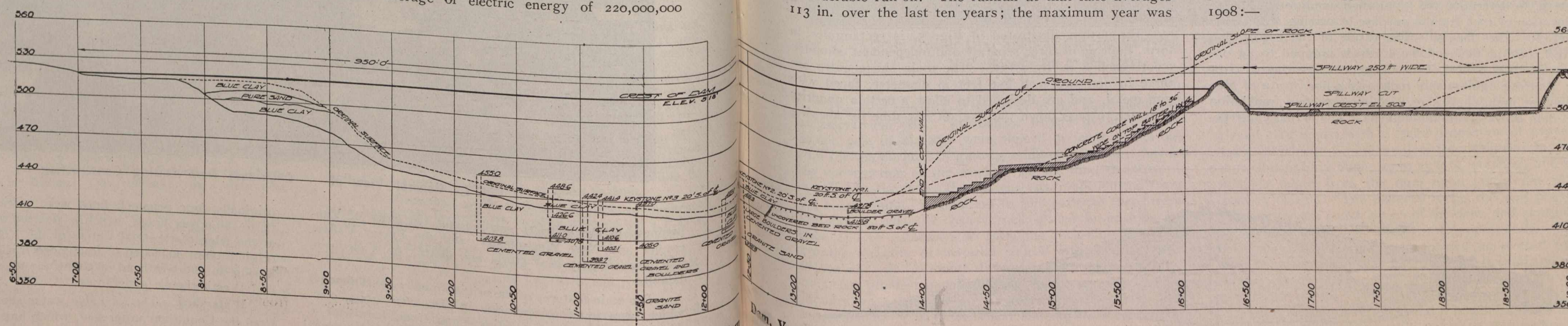


Fig. 3.—Profile Along Centre Line of the Coquitlam Dam, Vancouver Power Company, Limited.

1908-09	40,000,000	kilowatt-hours
1909-10	48,000,000	" "
1910-11	75,000,000	" "
1911-12	102,000,000	" "
1912-13	126,500,000	" "

It was therefore decided that the original plant which had been increased from time to time, bringing the total output up to 42,500 h.p., should, with the available storage in the Coquitlam Lake, be increased to a total of 83,000 h.p., so that the two plants could be operated satisfactorily as a peak load plant in conjunction with other plants contemplated by the company in the future.

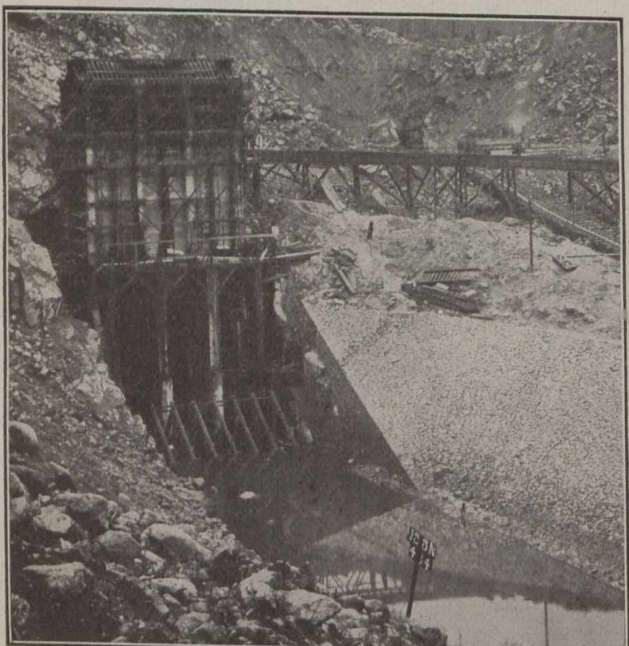


Fig. 5.—Gate Tower at Diversion Tunnel, with Flashboards for Temporary Water Storage.

New Dam.—A plan and profile of the new dam which is of the hydraulic-fill type with heavy rock toes, is shown in Figs. 2 and 3 respectively. The preliminary operations of building the dam were begun in the winter of 1908. Much exploration work was rendered necessary by opposition to the construction of the dam from various quarters, to determine the foundation conditions and also the location of rock for the purpose of driving a diversion tunnel for the floods. The result of the exploration showed rock in the east bank which dropped to an unknown depth one-third of the distance across the old river bed; abutting against this rock was a fine strata of blue and yellow impervious clay, making a perfect foundation for a dam of the hydraulic-fill type. The actual construction of the dam proper was not begun until March, 1912. The main dimensions of the new dam are as follows:—

Height of dam on centre line	99 feet
Extreme width of dam at base	655 "
Width at crest	40 "
Length of dam along crest, exclusive of spillway	950 "
Width of spillway	250 "
Elevation of spillway above sea level	503 "
Crest of dam above sea level	518 "
Slope of up-stream face	1 in 5
Slope of down-stream face	1 in 2 to 1 in 4
Original area of lake, old dam	2,328 acres
Area of lake at elevation 503 ft.	3,075 "
Storage capacity of lake above elevation 432 feet	192,100 acre-ft. or 8,369,000,000 cu. ft.
Storage capacity in electric energy at 78% efficiency	60,600,000 kw. hrs.

For the purpose of diverting the river and flood waters during the construction of the dam, a tunnel 490 ft. in length, having a clear width of 26 ft. with a height of 18½ ft. in the centre, was constructed on the east bank. This tunnel, which has its invert level at El. 435, was designed to carry 12,000 cu. ft. per sec. when the lake level was at El. 475, but the maximum quantity discharged through it did not amount during construction to more than 6,000 cu. ft. The maximum recorded flood over the old dam during the past ten years was 12,000 sec.-ft., or approximately 116 cu. ft. per sec. per sq. mi. of drainage area.

Gate House.—The gates for controlling the water through the diversion tunnel were placed in the concrete tower at the end of the approach channel. The bottom of the tower is at El. 455, spanning the tunnel entrance and supported on piers placed parallel to the floors in the tunnel. Girders were provided at the up-stream end of the piers for six temporary steel roller gates each 4 ft. 6 in. wide by 17 ft. high. These gates were controlled during the construction of the works by means of a gasoline hoist placed at El. 475 and arranged so that as the water rose in the lake they could also be operated at El. 508.

Upon the completion of the dam these gates were closed behind with a bulkhead of concrete filling up the whole space at the bottom of the tower, and forming a solid mass of concrete 26 ft. thick.

The tower, illustrated in Fig. 5, is rectangular in form, built up of three separate and independent compartments each 8 ft. by 11 ft. The main sluice gates are three in number, each five feet in diameter, faced with gun metal, and are designed to allow a certain quantity of water to pass down the river at any time for keeping

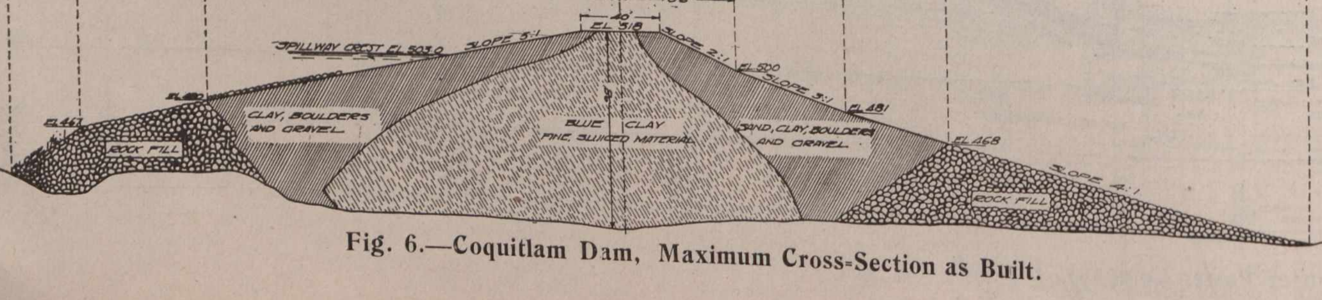


Fig. 6.—Coquitlam Dam, Maximum Cross-Section as Built.

it clean when required. These gates are operated at El. 513 within a gate house built at the top. The exterior openings to the three compartments are provided with auxiliary sluice gates made up of steel I-beams and timbers; but these will only be used when inspection of the main gates are necessary. Inclined racks are built against the front of these gates to protect them from water-logged timber and trash that might be drawn towards them when the gates are open.

In the construction of the works 1,150,000 cu. yds. of material were handled; this included the excavations for the approach channel to the diversion tunnel, and the outlet channel from the diversion tunnel and tunnel excavations.

The dam, as already mentioned, is of the hydraulic-fill type, the materials being sluiced by pumped water from borrow pits on the southwest and northeast banks of the river. Its cross-section is shown in Fig. 6.

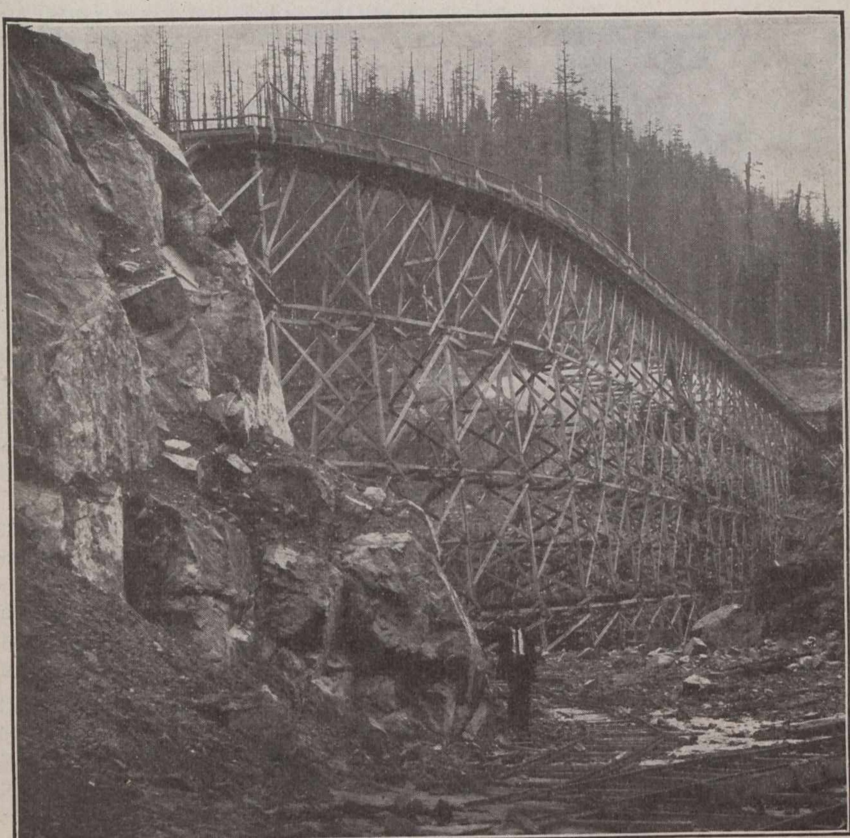


Fig. 7.—A 70-ft. Trestle Over Spillway Cut, for Carrying Flume from N.E. Borrow Pit to the Dam.

The total quantity of material in the dam amounted to 544,710 cu. yds. In addition to this quantity about 40,000 yds. of material were sluiced in front of the old dam for the purpose of gaining additional storage during the construction of the work. Of the total quantity of material in the dam, 489,800 cu. yds. were sluiced from the borrow pits. In addition to this material there were 116,360 cu. yds. of heavy rock placed in position by hand and by cableways to form the rock-fill toes. The average quantity of material sluiced into the dam was 47,400 cu. yds. a month from October, 1912, to July, 1913, the greatest quantity being 77,700 cu. yds. during January, 1913; an average of 2,500 cu. yds. per day. The total number of days in which sluicing was carried on with two shifts of ten hours each was 296 days, included in which time were periods necessary for removing flumes.

The main flumes were laid on an average grade of 4%. These were carried on trestles of various heights reaching to a maximum of nearly 70 ft. One of these is illustrated in Fig. 7. The flumes were formed of two 12 x 1-in. planks in width, and 2 ft. 6 in. high, lined on the bottom with 6-in. hemlock blocks, a 1 x 6-in. plank being nailed inside along each side of the flume. The trestles supporting the flumes were formed of 6 x 6-in. posts, caps and stringers for the first deck, with 4 x 4-in. posts, 4 x 4-in. caps, and 6 x 6-in. stringers for the second deck. These trestles were placed 16 ft. apart, and very satisfactory results were obtained from the type of flume adopted. The lining of the blocks in the bottom of the flumes from the northeast borrow pit was renewed only once during the construction of the works, and the remainder required occasional patching only.

Ball bearing giants, 4 in. and 5½ in. in size, fitted and controlled by the Hendy deflector, were used under a pressure of about 80 lbs. per sq. in. at the nozzle. These delivered for sluicing operations 215,000,000 cu. ft. of water during the actual dam construction. The percentage of solids to water carried from the pits amounted to 6.14%, representing 5.36% of solid material as measured in place in the dam. For conveying the water to the monitors, 16-in. flange pipes in 17-ft. lengths were used. The labor costs of sluicing ranged in different months from 6 to 16 cents per cu. yd., depending upon the amount of work to be done in removing flumes.

The pumping plant consisted of two Dayton centrifugal 3-stage pumps with 10-in. suction, 8-in. discharge, working at 150 lbs. pressure, rated to deliver 4 cu. ft. per sec.; two Byron-Jackson, 3-stage, centrifugal pumps, 10-in. suction and 10-in. discharge, rated to deliver 7½ cu. ft. per sec.; one Worthington 3-stage centrifugal pump rated to discharge 4 cu. ft. These were driven by five electric motors having a combined capacity of 1,125 h.p.

Power was delivered to the dam for construction purposes at 34,000 volts and transformed down to the required voltage at the works.

Two electrically operated Lidgerwood cableways of 3-ton capacity were used for depositing the rock toes of the dam, one of 1,100 ft. span and the other 1,200 ft.

The clay from the borrow pits for the construction of the dam was a very finely stratified blue and yellow glacial clay mixed with a large proportion of gravel and heavy boulders. Samples taken from this clay in the pits showed it to have about 23 to 25% of moisture, and actual samples taken from the dam upon completion showed from 25 to 27%. The material forms the very finest clay concrete for the construction of the dam, and is absolutely impervious. Fig. 6 shows the approximate position in which the materials were placed as plotted for the monthly progress diagrams. The up-stream and downstream slopes of the dam were very heavily rip-rapped with rock.

By reference to Figs. 6 and 8 it will be seen that the dam has been designed with unusually flat slopes and a wide crest. This is due to the fact that the stability of the dam had to be such as to satisfy the sentimental objections of the population living below the dam, although these dimensions were entirely unnecessary from an engineering point of view.

The spillway, which is capable of discharging over three times the maximum recorded flood, was excavated on the east bank of the river in solid granite, about 90,000 cu. yds. of material being removed and deposited by the cableways to the rock toes. Over the spillway a steel bridge of two spans, each 125 ft. in length, has been constructed to form a continuation of the roadway over the dam to the Westminster intake tower.

Westminster Water Supply.—Lake Coquitlam has, since the year 1892, been utilized as the source of water supply for the City of New Westminster. A company called the Coquitlam Waterworks Company, formed in 1886, now owned by the Vancouver Power Company, sold

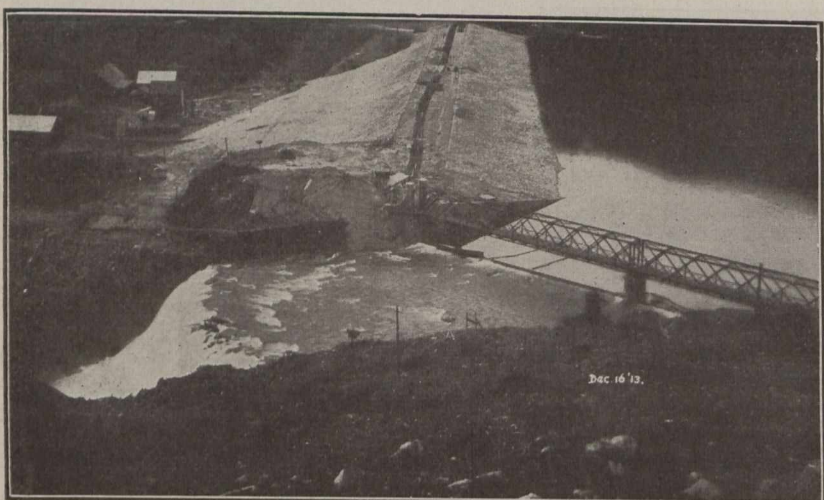


Fig. 8.—Coquitlam Dam and Spillway, Nearing Completion.

to the City of New Westminster in the year 1889 certain rights in the waters of Coquitlam Lake for the purposes of the city supply, and the works were begun by the city soon afterwards. This supply was first drawn from Lake Coquitlam in the year 1892, from an intake which was situated on the west bank of the river near what was subsequently the site of the old crib dam. From this intake a 14-in. steel pipe main was carried to a distributing reservoir in the City of New Westminster. This main was sufficiently large to supply water for that city until 1912.

Under the agreement entered into between the Vancouver Power Company and the Department of the Interior of the Dominion Government the company undertook to carry out certain improvements for the protection of the Westminster water supply, and among other things that were agreed upon was the provision of the new intake tower situated 1,000 ft. north of the old intake on the east bank of the lake. From this intake tower a tunnel 1,940 ft. long was constructed, the greater part of its length, being in solid rock 4 ft. wide by 7 ft. high, with a concrete invert. In other portions where the tunnel passed through cemented gravels and clays, it was formed of 48-in. steel pipe backed with concrete. The intake tower is a heavy concrete tower which has its foundations on rock. The outlet of the tunnel is at El. 428, or 4 ft. lower than the lowest draw-off of the Coquitlam-Buntzen power tunnel, while the floor of the gate house on the top is at El. 518, corresponding to the crest level of the new dam. The intake tower is circular in plan, and has an inside diameter of 18 ft. top and bottom. The walls are 4½ ft. thick, from the bottom to El. 465, and then taper to 18 in. thick at El. 518. A concrete arch bridge connects the intake tower with the roadway on the shore. In the walls of the tower there are four 40-in. square openings fitted with cast iron gates, and cast iron screens on the inside, the exterior openings being protected with wrought iron screens. These openings are placed at El. 430, 451, 469 and 487, distributed around the outer wall of the tower. The copper screens are arranged so that they can be raised for cleaning purposes to a floor inside the tower placed at El. 508. In addition to the control of water from the exterior of the tower, a secondary control is obtained within the tower so that the water may be drawn off at any desired elevation. This intake consists of a standpipe 42 in. in internal diameter, built up in four separate sections, each section having conical seats on the upper and lower ends, and each section fitting to the one next below it. The bottom section rests on a special cast iron elbow set in the base of the tower. These intake pipe sections are guyed to 60-lb. rails placed on opposite sides of the pipe, and bracketed to the tower wall at frequent intervals. Lifting rods 1½ in. in diameter are attached diametrically opposite near the top of each pipe section. The intake is operated by hand by means of a special lifting gear which may be attached to any set of lifting rods. The openings into the intake pipe are at El. 433, 451, 473 and 481. To form the approach to the tower a channel was excavated within a cofferdam 20 ft. wide at the bottom with side slopes 1½ to 1, heavily rip-rapped with rock.

For the protection of the Westminster water supply the company carried out extensive clearing operations on the shores surrounding the lake, which were covered with a heavy growth of cedar and hemlock. For a distance of over three miles above the intake, the whole of the land to be flooded was cleared and the stumps sawn close to the ground. The clearing over this section was completely done to El. 508, and in the upper part of the lake the whole of the shores have been cleared to El. 480. The total area of land completely cleared amounts to approximately 750 acres.

This work proved enormously difficult, owing to the steep sides of the lake and the necessity of constructing rafts plated with steel for burning much of the debris. The cost of clearing has been upwards of $600,000. The lowest cost for clearing was about $350 an acre, while sections of the work in swamps where the timber was decayed and heavy, cost as much as $2,000 an acre.

The Coquitlam Lake water is of wonderful purity and is almost sterile. The following is a typical analysis of the water:—

Water Analysis—October 18, 1913.

(Parts per million.)

Physical Examination—

1. Turbidity	None
2. Reaction	Neutral
3. Smell	None
4. Taste	Good
5. Sediment	Slight
6. Color	30

Chemical Examination—

1.	Ammonia, free, expressed as nitrogen	.056
2.	Ammonia albumenoid, expressed as nitrogen	.07
3.	Nitrates, expressed as nitrogen	.06
4.	Nitrites	None
5.	Chlorine	3.24
6.	Hardness, total, expressed as CaO	2.20
7.	Hardness, permanent	1.40
8.	Hardness, temporary	.80
9.	Oxygen consumed, 4 hours at 37° C.	2.7
10.	Oxygen consumed, 3 minutes at 37° C.	.4
11.	Solids, total	18
12.	Solids, volatile	8
13.	Solids, fixed	10
14.	Poisonous metals	None

Microscopic Examination—
Vegetable fibres and crystalline matter.

Biological Examination—

Number of bacteria per c.c.	30.
Presumptive test for color bacilli	Negative

Samples of water have been carefully analyzed fortnightly during the past four years, and the number of bacteria per c.c. has varied from 15 to 100, but has averaged throughout the whole period of the work about 40 or 50 per c.c. In spite of the fact that over 800 men were employed upon the works, of whom nearly 600 were employed on lake clearing alone during some months, no pathogenic organisms were discovered at any time, a result due to the very careful sanitary precautions which were taken in connection with the construction of the works.

Power Plant No. 2, Lake Buntzen.—Owing to the fact that suitable foundations could not be obtained for extending the existing power house of a sufficient size to contain the new units, a new site was decided upon about 2,000 feet south of the existing power house No. 1, and there a new and independent plant has been constructed. The essential features of this new plant are shown in Fig. 9.

Water is obtained from Lake Buntzen through a concrete-lined tunnel 14 ft. 8 ins. internal diameter, and about 1,800 ft. long, driven through solid rock.

At the upper end of the tunnel three 6½-ft. Doble intake valves are provided, which are operated by oil pressure.

In order that better speed regulation of the machines may be obtained, and that the effects of water in the pipe lines, due to sudden changes of load may be reduced, a surge tank is provided at the tunnel portal. This surge tank is 30 ft. in diameter and about 90 ft. high, and is built of riveted steel plates.

The pipe lines, of which there are three, are connected to the surge tank by means of reinforced plates. The upper ends of the pipes project into the surge tank, and are each provided with a bell-mouth, thus minimizing entrance losses. The pipes are 8 ft. 6 in. diameter and ½ in. thick at their upper ends, and taper to 7 ft. diameter at the power house, where the thickness is 1⅛ in. About 200 ft. from the power house the pipe lines pass into tunnels driven through rock, which is badly fissured. The slopes range from 28 to 53 degrees. A short distance below the surge tank, a Doble venturi butterfly valve is provided on each pipe line.

The power house building (Fig. 10) is of reinforced concrete construction, and is founded throughout on solid rock. On the main floor, 5 ft. above high-water mark, three hydro-electric units of a combined capacity of 40,500 h.p. have been installed. Each unit consists of one Dick Kerr 8,900-k.v.a., 3-phase, 60-cycle alternator, generating current at 2,200 volts, direct-driven at a speed of 200 r.p.m. by four Pelton-Doble water-wheels of the impulse type, the combined capacity of which is 13,500 h.p. The rotor of the alternator and the four water-wheels are all pressed onto a hollow nickel steel shalt 51 ft. 3 ins. long, and operated in one piece.

At the power house each pipe line divides into four branches, each branch supplying water to one wheel of the unit. On each branch a Doble hydraulically operated gate valve is provided, which controls the admission of water to two needle nozzles, which direct the water to the buckets of each wheel.

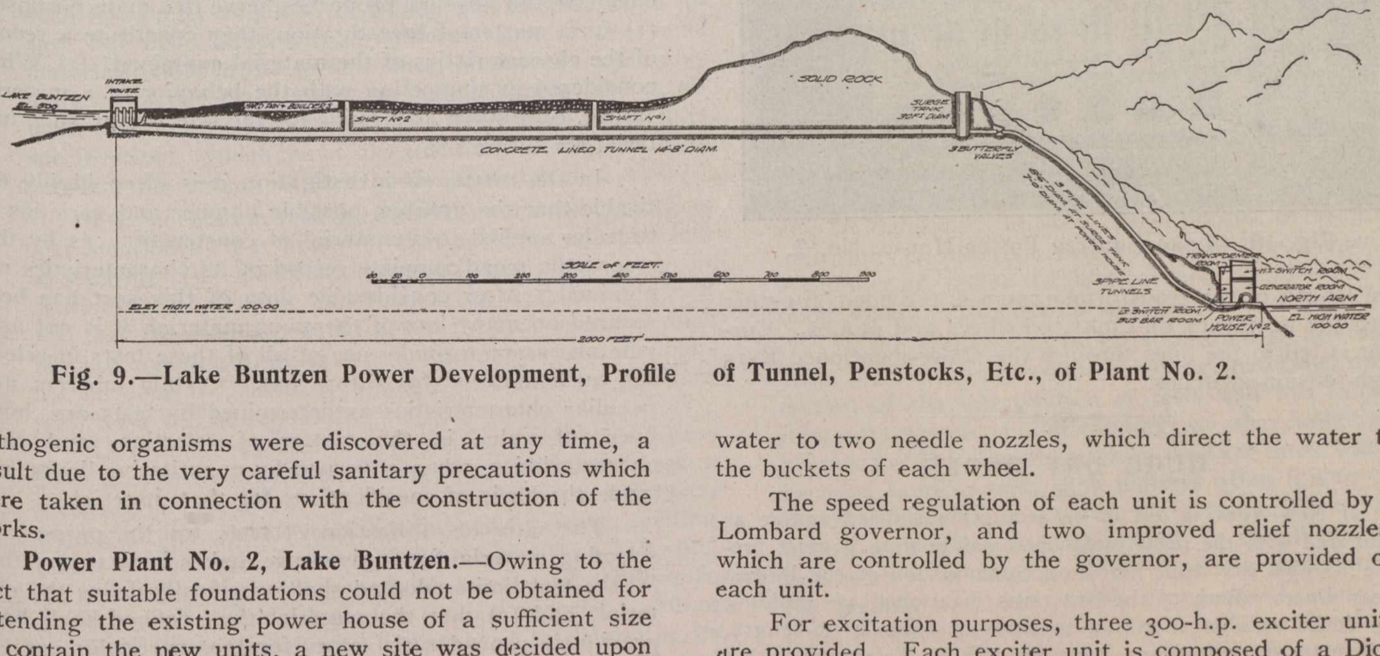


Fig. 9.—Lake Buntzen Power Development, Profile of Tunnel, Penstocks, Etc., of Plant No. 2.

The speed regulation of each unit is controlled by a Lombard governor, and two improved relief nozzles, which are controlled by the governor, are provided on each unit.

For excitation purposes, three 300-h.p. exciter units are provided. Each exciter unit is composed of a Dick Kerr induction motor generator set, direct-driven by two Pelton-Doble water-wheels mounted on the end of the shaft. The speed of the exciters is 600 r.p.m.

The excitation voltage is 250, and the voltage regulation on the A.C. bus bars is controlled by Tirrell regulators.

Owing to the great size and weight of some parts of the units, two 50-ton electrically operated travelling cranes are provided. These cranes control the entire length of the main generator room.

Immediately above the generator room is located the high-tension switch room, which also contains lightning arresters, etc. The high-tension switches are of the C.G.E. K-15 type, and are suitable for 60,000 volts. The lightning arresters are of the 4-tank electrolytic type.

To the rear of the generator room and above the bus bar compartments is the low-tension switch room; the low-tension switches are all of the C.G.E. H-6 type.

The transformer room is located immediately above the low-tension switch room. The transformer equipment consists of four banks of three 3,000-kw. single-phase, 60-cycle, oil-insulated, water-cooled transformers, by means of which the voltage will be raised from 2,200 to 34,600. At a later date the transformers will be "Y" connected, thereby increasing the voltage to 60,000. One 25-ton electrically operated travelling crane which travels the entire length of the transformer room is provided, together with the necessary oil tanks, oil filters and pumps. Current is fed to the lines through the above-mentioned K-15 high-tension switches.

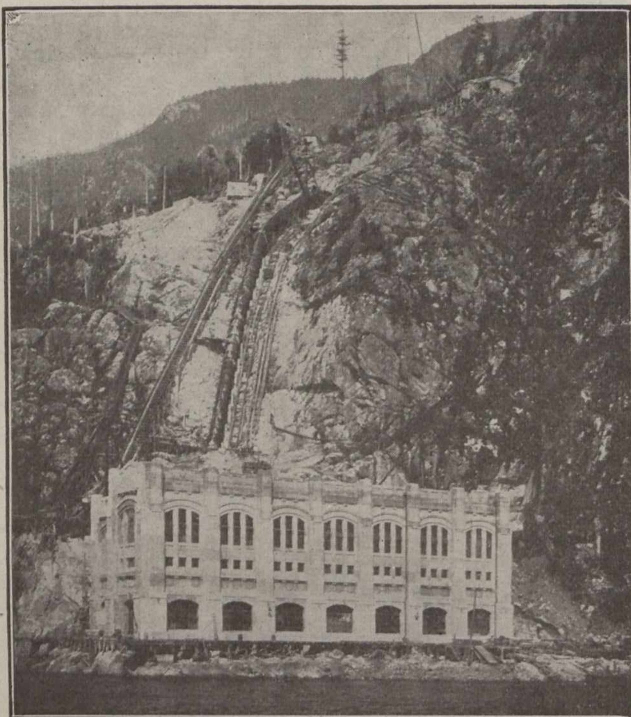


Fig. 10.—Lake Buntzen Power House, No. 2.

HUGE DRY DOCKS.

If New York is not to be at a serious disadvantage as compared with the ports of Quebec and Boston, it must provide its own dry dock for the accommodation of the largest ocean liners. Both of the two ports mentioned are building dry docks capable of taking in ships of 1,000 ft. in length. It is interesting to learn, says the "Daily Telegraph," that the New York Dock Commission is planning to build a dock of this size at South Brooklyn. The dry dock to be built at South Boston is to cost $3,000,000, and a contract has been entered into by which the International Marine, the Cunard and the Hamburg-American lines will pay $50,000 a year for 20 years for its use. The announcement of the intention of the Nova Scotia Government to erect at Halifax a huge dry dock, one capable of receiving the largest merchant vessels and warships afloat, or, as it is stated, likely to be afloat, is certainly indicative of what the expectations of the business of the port are. It may be added that the large graving dock which the United States Navy Department is building at Pearl Harbor will be finished as originally planned. The chief difficulty in constructing the dock, which has now been overcome, is the shifting of the sand bottom.

THE TESTING OF BITUMINOUS MATERIALS FOR ROAD AND STREET CONSTRUCTION.

ONE of the papers presented at the 10th annual convention (Philadelphia, Dec. 9 to 12, 1913) of the American Road Builders' Association dealt chiefly with the testing of bituminous materials to be used in the construction, maintenance and repair of roads and streets. It outlined clearly the fundamental principles upon which such tests are based, and brought out the importance of the relation of these tests to paving specifications. The paper was read by Mr. Prevost Hubbard, of the Institute of Industrial Research, Washington, to the "Construction" division of the convention. What follows here constitutes an abstract of Mr. Hubbard's remarks. The reader is also referred to a discussion of the paper by Mr. Francis P. Smith, of Dow and Smith, chemical engineers, New York, which appears on another page of this issue.

For reasons that are not clearly apparent, the fundamentals of testing, in so far as they relate to the practical utilization of tests, have in many cases been overlooked by testing engineers and chemists in connection with bituminous road and paving materials. Disregard of these principles has undoubtedly created more confusion and misunderstanding on the part of highway engineers and contractors than any other one cause.

As applied to materials of construction, tests of both chemical and physical properties serve two main purposes. (1) As a matter of investigation, they constitute a record of the characteristics of the material examined. (2) When considered in connection with the behavior of a material in use, they serve as a guide for the selection of such material for future use.

In the matter of investigation it is often highly desirable that the greatest possible number and varieties of tests be applied to a material of construction, as by this means the most complete record of its characteristics are obtained. After considerable data of this sort has been secured on many lots of the same material, it is not as a rule necessary to make use of all of these tests in selecting such material for future use. Certain inherent and peculiar characteristics as determined by tests are, however, of value in the matter of selection, and such characteristics, when governed by quantitative limits, are made the basis of specifications for that material.

The ultimate utilization of tests for the purpose of selecting material for a given use makes it necessary that (1) the test limits adopted shall specifically define the material, and (2) that the material thus defined shall have previously proved satisfactory for that particular use.

For some classes of material these points are not difficult to cover. In the case of bituminous materials, however, the matter is complicated by the numerous varieties or types of bitumen in common use, and the overlapping characteristics of various grades of these different types. The interpretation of tests in general is therefore no simple matter, and numerous misconceptions of the value of certain tests are prevalent. Standards of more or less established test values have been thrown into question by the introduction of new types of bituminous materials which, while similar to or identical in many respects with other better known materials, yet differ from them materially in certain physical or chemical properties. This has made it necessary to either modify old standards or to create new standards to be used specifically for the new materials introduced. It is the author's opinion that the latter method is in general to be preferred for reasons that will appear later.

The individual tests required by specifications for bituminous road and paving materials may serve one or more of the three following purposes: (1) They may directly indicate the suitability for a given use, of the material specified. (2) They may serve as a means of identifying the source of a material or even the material itself. (3) They may serve to control uniformity in the preparation or manufacture of a material.

The first of these purposes is undoubtedly the most important and is usually the only one considered by the lay mind. In the case of bituminous materials, this purpose is only partly accomplished by a comparatively few tests. As examples may be mentioned tests of constituency, such as the penetration test, the float test and the test for viscosity. Such tests can only be of maximum value, however, when applied to a specific type of bituminous material and when considered in connection with other tests, which, by themselves, may not directly indicate suitability. Thus, for a certain type of bituminous concrete pavement the proper penetration limits at 25° C. for a California asphalt may lie between 7.0 and 9.0 mm., while the proper penetration limits for a fluxed Bermudez asphalt to be used in exactly the same type of pavement and under the same conditions may be entirely different, say from 14.0 to 16.0 mm. It is evident that to attempt to cover the penetration limits for both materials under one specification would be useless. In the first place, such test limits as 7.0 to 16.0 mm. are so wide as to insure but little uniformity in different lots of the same materials; and, in the second place, an entirely unsuitable material of one class might be supplied under the maximum test limit of the other class. The fallacy of blanket specifications, which have already been advocated to a considerable extent, is thus easily demonstrated.

If a penetration test is essential under the conditions just mentioned, it is apparent that recourse must be had to separate type specifications; and if this is so the specifications must contain either tests or test limits which will describe certain peculiarities of the type specified, that are not common to other types. In many cases this cannot be done by means of a single test and two or more such tests will be required.

This brings us to a consideration of the second purpose previously mentioned, i.e., the use of tests as a means of identification. There are a number of such tests, among which may be mentioned specific gravity, melting point, solubility in carbon disulphide, fixed carbon, etc. So far as the usual test records are concerned, the specific gravity of a bituminous road or paving material is one of the most important characteristics used to determine its identity, and this is particularly true if its specific gravity is considered in connection with the consistency of the material and sometimes its solubility in carbon disulphide. Thus a bituminous material with a specific gravity of 0.99 and penetration of 7.0 mm., at 25° C. must be a blown product. Fluid consistency and high specific gravity, say 1.25, in a tar serves to identify it as a coal tar, and the identification is strengthened if its solubility in carbon disulphide is low, say 75 per cent. Numerous other examples of a similar nature might be cited and a treatise might be written upon the value of tests by themselves and in relation to other tests as a means of identifying bituminous materials.

High fixed carbon in most asphalt cements produced from Mexican petroleum is a distinguishing characteristic. Relatively low fixed carbon in good asphalt cements of the same consistency produced from California petroleums serve to differentiate them from the Mexican products. Here, again, the necessity or desirability of different test limits are apparent, for if the amount of fixed carbon yielded by a California asphalt cement was as high as the 16 per cent. often found in Mexican asphalt cements, indications would point very strongly to injury of the former due to excessive temperatures having been employed in the process of manufacture.

This leads us into the third purpose for which tests may be made to serve; control of uniformity in the preparation or manufacture of a material. Among such tests may be mentioned those for determining flash point, loss by volatilization, distillation, solubility in given grades of paraffin naphthas and solubility in carbon tetrachlorides. Practically all of the other tests previously enumerated may also be made to serve this end. No one by itself will, however, necessarily accomplish this purpose, no matter how close the test limits are drawn. This is mainly due to the fact that products of innumerably varied and complex characteristics may be produced from a given crude material by modifying the methods of manufacture.

Thus, by direct distillation of a given crude petroleum, an asphalt cement of 10.0 mm. penetration can perhaps only be produced by the removal of a considerable amount of distillate and the application of comparatively high temperatures. If distillation is discontinued in an intermediate stage, however, and the blowing process employed, an asphalt cement of the same penetration may be produced with the removal of much less distillate and the application of a lower maximum temperature. In the second case, the resulting product, while of the same consistency as the first, may have a lower specific gravity, a higher melting point, a greater penetration at low temperature, and a less penetration at high temperature. Other properties such as fixed carbon, naphtha, soluble bitumen, loss by volatilization, etc., may also be different. In such cases, control or assurance of uniformity in different lots of material must depend upon a number of tests, the interrelation of which is clearly understood, and for which suitable limits are specified.

In the preparation or interpretation of any specifications for bituminous road or paving materials, an appreciation of the interrelation of tests and test limits is as necessary as an understanding of the individual significance of the tests themselves, and yet those who should be most familiar with such matters often fail to consider the possible relations which a given test may bear to others with which it is associated in specifications. The interrelation of tests and test limits is something which the layman may not readily comprehend, and this has often resulted in his innocent acceptance and enforcement of unjustly discriminative specifications prepared or suggested by those who have an object to attain.

In the author's opinion, discriminative tests and specifications are perfectly proper and often desirable, if used in the right manner. They are, in fact, necessary to use, unless a more or less valueless blanket specification is adopted. When so used, however, their significance should be clearly apparent and they should not be put forward as open specifications.

When a given type of bituminous material has a single characteristic property which distinguishes it from other types, test limitations of this property may be so drawn as to make a specification discriminative. The status of such specifications is not usually difficult to ascertain. Discrimination is, however, sometimes secured by the use of a combination of two or more test limitations, the significance of which is only apparent under the close scrutiny of one who has an intimate and comprehensive knowledge of all types of bituminous materials. Specifications of this class may, as a whole, absolutely eliminate competition, although no single clause in the specifications could be criticized from this standpoint.

If competition is eliminated by a single specification, as is sometimes advisable or even necessary, in order to insure a satisfactory product, it may often be restored by the use of two or more specifications which will serve as alternatives. When this is done, two or more types of bituminous materials will be specified, which are of equivalent value in so far as their suitability for a given use is concerned. Thus while different test limitations and sometimes different properties are covered by the different specifications, each particular combination of test limitations and properties, constituting a given specification, will be considered as equivalents.

From the foregoing discussion, it must be evident that the preparation of specifications for bituminous road and paving materials is often a complicated matter and should only be undertaken by one who has thoroughly familiarized himself with the origin and manufacture or preparation of all types of bituminous materials as well as with their physical and chemical characteristics. It is the author's experience that comparatively few highway engineers in this country are sufficiently acquainted with these materials to warrant them in preparing such specifications. Many who have attempted the task have failed because of lack of accurate information which has led them to combine and make use of portions of specifications prepared by others. Such combinations have most often proved utter failures, some of which are ludicrous, inasmuch as they have actually defeated the object for which they were presumably drawn. Others have been impossible because of the fact that certain clauses or test limitations prescribed were incompatible.

In this connection it may be said that while the average highway engineer will not find it practicable or even necessary to also become a highway chemist, he should nevertheless possess as a part of his practical working equipment some knowledge of the chemical and physical properties of bituminous materials and methods of testing them. A lamentable lack of such information is apparent in many highway engineers who, in other branches of their profession, are thoroughly capable and efficient. Opportunities for obtaining instruction along this line are now being offered by at least one of our universities, and many engineers should find that the comparatively short time required for this purpose could not be spent to better advantage.

One other point may be mentioned regarding the preparation of specifications which is directly connected with test limitations, and this is the matter of allowable variations in results which may be looked for from different chemists and different laboratories. In the first place it should be remembered that at the present time there are comparatively few standard methods for making tests which have been generally adopted. Variations in results are frequently attributable to variations in methods of testing. It is, therefore, good practice to include in, or as a part of specifications, descriptions of the methods to be employed in testing. This is especially true where specifications are to be widely used.

Besides the above mentioned cause, a certain variation in results may be expected from what is termed the personal question. Thus, no matter how clearly defined a method may be nor how conscientiously followed, it is the exception rather than the rule when two operators working on samples of the same material obtain exactly the same results. In fact, a single expert operator will seldom check himself exactly, although his results may be identical in so far as their practical application is concerned. Moreover, it should be realized that in substance as complex in character as are bituminous materials, however carefully prepared or manufactured, there is apt to be some variation in samples of the same batch whether taken from the same still, tank, kettle, barrel or even from the same sample can.

Just what the allowable variation in results should be when all of these conditions are taken into consideration, is a lengthy matter to discuss. It is certainly not the same for all tests, nor even for the same test applied to different classes or grades of material. Thus in the ordinary volatilization test a variation of 1 per cent. in results obtained upon a material losing 15 or 20 per cent. would be perfectly reasonable, while in a material losing 2 per cent. it would be an inexcusable variation. Owing to lack of time, the author does not consider it advisable to attempt a detailed discussion of this subject in the present paper, but suggests it is a topic for consideration.

In conclusion, the author wishes to state that while this paper may appear to deal with specifications in general more than with the actual testing of materials, it should be remembered that specifications are, in effect, definitions, and that from a broad standpoint definitions are themselves fundamental tests.

TRENT VALLEY FOREST RESERVE.

The Commission of Conservation is recommending that the 2,000 square miles of land in the Trent Canal watershed be set aside by either the Dominion or Provincial Governments as a forest reserve. Such a protective measure would under the peculiar circumstances, seem to be well advised. The water supply for the Trent Canal and Kawartha Lakes is obtained from the watershed mainly, and would be seriously impaired if the area in question were denuded of the remaining timber. Since the Dominion Government in 1905 obtained control of the water rights, much valuable work in damming up the back lakes has been done, with the result that the flow of water for power purposes north of Peterborough has enlarged most satisfactorily. The Dominion has spent over ten millions already on the Trent Canal, and by the time the extensions to Georgian Bay and Lake Ontario are completed it will have spent several millions more. And further, there are cogent reasons beyond the water-conservation interest to support the commission's recommendation. Half of the area is now unpatented, or in possession of the Crown. Only a tenth of the land can be farmed, and the farming at the best is so poor as to be quite unprofitable. Less than 700 acres of a million acres of forest is untouched virgin area. Under the provincial order-in-Council of 1905, conveying to the Dominion the water control, the right to buy land along the lakes and water courses at 50 cents an acre was granted the Dominion Government, but only two thousand acres were bought. It seems obvious that the best interests of the district, as well as the requirements of a wise conservation policy, would be served by making the areas in question a reserve for afforestation purposes

The production of coal in Italy in 1912 was 663,812 metric tons, of which 660,491 tons are classed as lignite. Imports were 10,057,228 tons; exports, 26,288 tons. The coke made —largely from imported coal—was 1,223,902 tons; briquettes, 874,365 tons.

DISCUSSION ON MR. PREVOST HUBBARD'S PAPER, "THE TESTING OF MATERIALS FOR ROAD AND STREET CONSTRUCTION."

By Francis P. Smith.

WHILE the speaker agrees with much that Mr. Hubbard has said, he does not consider that his objections to the so-called blanket type specifications are valid, more especially with respect to asphalts. The asphalt paving industry is by no means a new one. For upwards of forty years pavements and roadways of this type have been in use in this country. The speaker has for eighteen years been closely identified with this industry, and during that period has had charge of the mining and refining of asphalts and the laying of bituminous pavements of all kinds, and he, therefore, believes that he is qualified to judge, and justified in stating that the requirements of an asphalt for paving purposes are well understood and are comparatively simple and have nothing to do with the sources of the material. For specification purposes he considers that so-called identification tests are unnecessary.

The function of an asphalt cement or asphalt binder is to cement together the particles of the mineral aggregate which forms the roadway. In order to do this it must be possessed of sufficient cementitiousness or binding value. If it fails in this, it is useless as a cementing material. It must be sufficiently pure; i.e., contain sufficient bituminous binding material, to make it available for use. It must be of such a consistency that it can be properly applied to the mineral aggregate in such a way as to thoroughly coat each particle of it. It must not be too susceptible to changes in temperature; i.e., become too hard in winter or too soft in summer. It must not harden too rapidly when heated in the melting kettles and when exposed to the elements it must maintain its original properties for a sufficient length of time to give satisfactory service in the pavement or roadway. All of these properties are determinable by well known tests, and these tests must be met by all asphalts before they can be considered suitable for paving or road making use. It is true that different asphalts vary in their properties. Some are purer than others; some are more susceptible to changes in temperature or to prolonged heating, and some have higher cementing value than others. Experience has shown that a suitable asphalt for paving purposes need not possess all of these qualities in the highest degree, but sufficiently so for practical purposes. From past experience, however, it is perfectly possible to clearly define minimum limits which all asphalts must pass in order to be accepted with safety for paving work. Having done this, you have established the ideal specification of the blanket type which calls for all the necessary qualities and does not differentiate as to source, but as to quality only, and is not of excessive length. This fulfils the first purpose of a specification for bituminous road and paving materials as defined by Mr. Hubbard. Except in special cases, just why a specification should state at great length a number of tests for identifying the material, as stated by Mr. Hubbard to be the second purpose, the speaker fails to see. Anyone really competent to test bituminous materials for paving or roadmaking purposes can identify them just as easily without a specification as with one and certainly the contractor does not require this information to enable him to bid intelligently.

As to securing uniformity in the preparation or manufacture of a material, it would seem to be a simple task to insert a clause in a blanket type specification stating the maximum permissible variation in different shipments of the same class of material. To further insure that one manufacturer shall not adulterate or lower the quality of his material in any respect or respects so that it will **just** meet the minimum requirements of the specification, it may be required that all shipments of material shall be fully equal to the established standard and recognized quality of that particular brand.

Mr. Hubbard uses the variations in desirable consistency of different asphalts as an argument against blanket type specifications. Assuming that the wide variations which he cites are altogether normal, these are by no means wholly dependent upon the source of the asphalt. The considerations which affect desirable consistency or penetration are:—

1. Purity.
2. Susceptibility to changes in temperature.
3. Character of mineral aggregate.
4. Climatic and traffic conditions.

As to purity, the fluxed Bermudez which he cites contains about 96 per cent. of bitumen, the California about 100 per cent. of bitumen. There is too little difference in this respect to afford an excuse for separate specifications.

As to susceptibility to changes in temperature, the California is much more affected by temperatures up to 140° F.; i.e., softens more readily but, on the other hand, loses less when heated to 325° F.

The character and grading of the mineral aggregate, however, have as great an influence on the desirable penetration as the susceptibility of the asphalt to temperature changes.

The climatic and traffic conditions for any one particular piece of work will be the same in each case and may, therefore, be dismissed from consideration.

Until the material is actually assembled, however, it is impossible to state with certainty just what will be the proper penetration or consistency of the asphalt cement or binder, as it is impossible in most cases to draw a specification which will call for a mineral aggregate possessing a predetermined and definite degree of stability, and upon this the desirable consistency of an asphalt cement or binder largely depends. This consistency is something which the trained and experienced engineer can and should determine while the work is in progress. While this argument applies more forcibly perhaps to sheet asphalt pavements than to asphalt macadam, it is neverthelesse true of both types of construction and may be readily met by a clause in the blanket type specifications to the effect that the exact degree of penetration or consistency within the limits established by the specifications shall be determined by the engineer in charge, depending upon the kind of materials used and the traffic upon the roadway to be paved.

On the other hand, let us assume that a separate specification is to be prepared for each kind of asphalt. Those commonly in use are prepared by refining or fluxing, or both, Gilsonite or crude asphaltic material obtained from California, Cuba, Mexico, Texas, Trinidad and Venezuela. All of these differ essentially in purity and susceptibility to changes in temperature, and from the standpoint of consistency or penetration should, therefore, according to Mr. Hubbard, be considered separately. This would make seven separate specifications. In the case of Gilsonite, the character of an asphalt cement or binder made from it would depend entirely on the kind of flux used with it, so that Gilsonite products alone might require two or more specifications. The same is true from the road binder standpoint of Trinidad, Cuban and Bermudez, while with California and Mexican asphalts the method of distillation used greatly affects the character of the product. In fact, the New York City paving specifications call for a special type of California asphalt much less ductile and susceptible to temperature changes than the standard California paving asphalt which has been successfully used for many years throughout the United States and is exclusively used on the Pacific Coast. Again, what is to hinder any manufacturer from combining two or more of these asphalts, as has been successfully done in many cases? To carry the separate specification idea, therefore, to a logical conclusion would involve the writing of an encyclopedia which would need constant revision and to separate the materials into two or three classes would be a poor compromise, leading to invidious distinctions, in many cases perhaps even to unfair discrimination, as it has done in the past. Such a compromise would, in the speaker's judgment and experience, be far worse than anything which has been alleged against the blanket type of specification.

In the early days of the paving industry closed specifications and high prices were the rule. As knowledge and experience in the paving industry have been acquired by engineers, closed specifications have become more and more rare and under proper blanket type specifications price has declined and quality has been maintained. The type brand of specifications were tried and abandoned as too cumbersome and the speaker freely admits that some years ago he advocated and drew a number of such specifications, but later became convinced that they were unnecessary and cumbersome and so abandoned them.

The Fixed Carbon Test.—With regard to the fixed carbon test,* the speaker considered that this is solely an identification test and therefore has no place in a specification unless it is desired to call for a particular kind or type of asphalt. This, as Mr. Hubbard states, may be perfectly proper in certain instances, but not in a specification where open competition is desired.

Many excellent pavements have been laid with California asphalt containing 16 per cent. and even 17 per cent. of fixed carbon, and it may, therefore, be considered at least doubtful whether the presence of this amount of fixed carbon in California asphalt is a sign that it has been injured in the process of manufacture.

The specific gravity test is undoubtedly of value as an identification test, but it is perfectly possible to recognize a blown asphalt without having recourse to this test, as, for instance, by the use of the ductility test. It is also questionable whether a certain amount of blowing is not the reverse of injurious so long as the ductility or cementing value is not reduced too greatly. Air blowing during distillation reduces susceptibility to changes in temperature and within limits and for certain purposes this is highly desirable.

A word with regard to the paraffine test.† The speaker believes that it has been conclusively proven that the presence of paraffine scale per se does not necessarily injure asphalt for paving purposes up to, say, 10 per cent., and possibly beyond that. The advocates of this test have recently taken the position that it is valuable as indicating the presence of an undesirable amount of paraffine hydro carbons in an asphalt, oils consisting largely of paraffine hydro carbons being recognized as unsuitable for the production of asphalt. To dispose of this claim it would seem to be only necessary to cite the case of many old and excellent pavements laid with Trinidad and other asphalts which contained over 25 per cent. of a flux composed almost wholly of paraffine hydro carbons. If 25 per cent. of paraffine hydro carbons is not injurious, where is the line to be drawn? It seems only reasonable to relegate this test to the class in which it really belongs, viz., the identification class, and rely on tests measuring the essential qualities of an asphalt when it comes to determining its fitness as a paving or road making material.

With regard to the ductility test, this has frequently been criticized by those who apparently do not understand its true scope or function. It has been claimed that the most ductile materials at ordinary temperatures, say 77° F., are those which are least ductile at low temperatures. This is not true. Great care must be taken in making ductility tests at low temperatures, otherwise the briquette will crack and no ductility will be reported. What is true is that the most highly ductile materials lose relatively a greater proportion of their ductility at low temperatures than do low ductile materials, but it must be remembered that even at 77° F. the latter class have practically no ductility to speak of. In the speaker's experience and in that of many others, the ductility test more nearly measures the cementing value of an asphalt than any other test known, and it is in that that its chief value lies. In this connection I will quote some remarks recently made by Mr. C. N. Forrest, of the Barber Asphalt Paving Comany, concerning the ductility test:—

"There is another fallacy in connection with the ductility test. It has been asked why should not the material be ductile at thirty-two degrees Fahrenheit, the supposition being that when a pavement expands and contracts, the resistance of the asphalt, under those circumstances, is dependent more or less on ductility, but the ductility test as performed according to the general way of doing it, as we know it, is a measure of the cementitious value. To produce a material that is ductile at thirty-two degrees Fahrenheit is to produce a short material; that is, if it is ductile at 32, it is not ductile at 77, and that would be done by the same method as increasing the toughness—by using a blown oil.

"The ductility test has nothing whatever to do with the capacity of the asphalt to give and take with the expansion and contraction. As a matter of fact, the asphalts which are rubbery and show some resistance to impact and show some ductility at a low temperature, will seldom show enough for sheet asphalt. They will crack in cold weather, although they are rubbery and show resistance to temperature changes."

As to variations in results as mentioned by Mr. Hubbard, the writer has often seen determinations reported in hundredths of a per cent. when the method employed was not accurate within tenths of a per cent. In similar cases he has also known material to be rejected because it exceeded the specification limits by two-tenths of a per cent. where the method used was not accurate within one-half of one per cent.

In conclusion, the speaker wishes to most heartily concur in Mr. Hubbard's statement as to the necessity

*See The Canadian Engineer, Vol. 25, pp. 703, 738, 780, 801, 808, and 872.

†See The Canadian Engineer, Vol. 25, p. 838.

for an understanding of the individual significance of the various tests and the relations of the different tests to each other. Without this knowledge the preparation of proper specifications and the interpretation of them is impossible.

INFLUENCE OF THE POSITION OF CROWN HINGE ON THE WEIGHT AND DEFLECTION OF A THREE-HINGED SPANDREL-BRACED ARCH.

THE form of three-hinged spandrel arch most frequently used in bridges is that in which the crown hinge is placed in the lower chord. Often, however, the crown hinge is located midway between the upper and lower chords, or in the upper chord, the weight and deflection of each member varying with different positions of the hinge. Just what the variation is in each case is given in a brief form by Messrs. M. A. Beltaire and R. W. Parkhurst in *The Cornell Civil Engineer* for December, 1913. Their results are from a series of investigations to compare the weights and deflections of three-hinged metallic arches as influenced by the shifting of the crown hinge from its ordinary position in the lower chord, to a position intermediate between the chords, and finally to the upper chord. These three positions are designated in the following as Case B, Case I and Case T respectively.

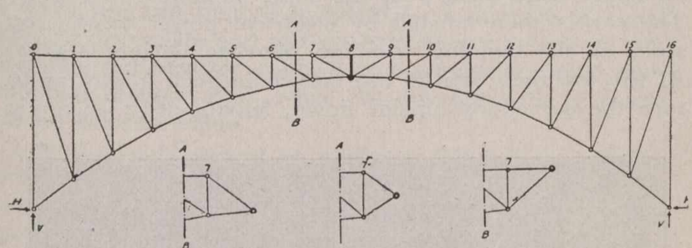


Fig. 1.—Three-Hinged Arch Bridge.

In conducting these investigations it was not thought necessary to work out a new design of a three-hinged metallic arch, a sufficient field for investigation being furnished by a study of the design of the Niagara spandrel-braced arch. This arch has a main span 550 feet long, supporting at each end a trussed span of 115 feet. The form of the main arch is as shown in Fig. 1. Each truss has a batter of 1 horizontal to 10 vertical, with a width between the axes of the upper chord of 30 feet; this makes the axes of the rib 34 feet apart at the crown and 56 feet 7.75 inches at skewback centres. The axis of the rib at the centre is 113.9 feet above the skewback line as measured in the plane of the truss.

Influence on Weight of Truss.—The investigation of the influence of the position of the crown hinge on the weight of the structure, conducted by Mr. Beltaire, was carried out by computing the maximum, minimum and reversed stresses for each member graphically and then computing the required section areas. The area for dead load was found separately from that for live load and to the latter was added 80% of the reserved stress.

In computing the weight of members, the theoretic weight was used, i.e., the weight as computed from the formula

$$W = 3.4\,A\,l$$

in which W = weight of member; A = area of section in square inches; l = length of member in feet and 3.4 = weight of steel bar one foot long and one square inch in cross-section. This ignores all latticing and details at joints and hence it is but a very close approximation; experience shows that this theoretic weight is slightly less than the actual weight, the ratio of actual to theoretic weight averaging about 1.25.

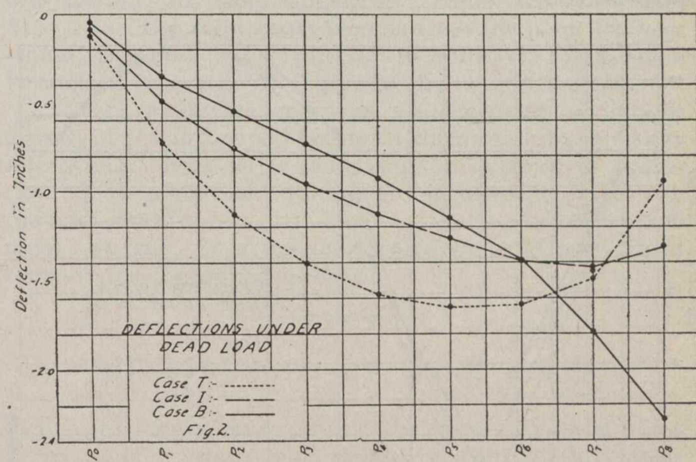


Fig. 2.

The weight of the structure in Case B was found to be 3132.96 kips, of which 12.9% was for the weight of upper chord members, 43.4% for lower chord members, 20.1% for the diagonals, and 23.6% for verticals. The weight in Case I was 2857.616 kips, as follows: Upper chord members, 13.2%; lower chord members, 36.9%; diagonals, 24.8%, and verticals, 25.1%. The weight in Case T was 2762.588 kips, as follows: Upper chord members, 19.1%; lower chord members, 32.8%; diagonals, 23.3%, and verticals, 24.8%. The weight in Case T was 88.17% of that of Case B and Case I 91.21% of that of Case B.

Conclusions.—From the above it is seen that Case T is the most economical design where a three-hinged arch is to be erected, it requiring 11.83 per cent. less material than Case B and 3.4 per cent. less than Case I. In a structure of the magnitude of the arch used for the computations, the economical advantage would amount to

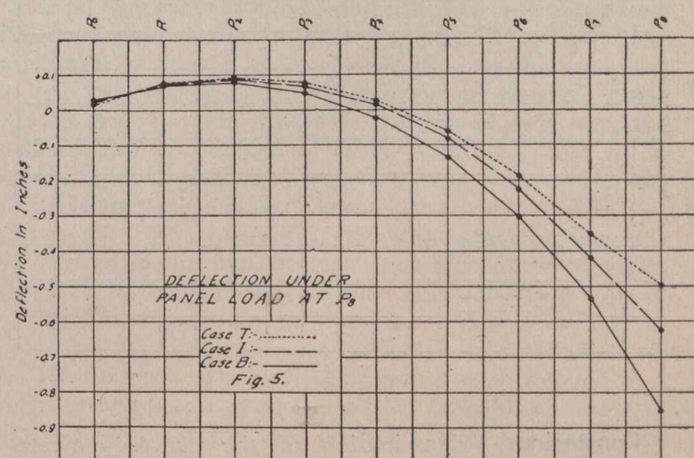


Fig. 3.

about $11,000, or $1,000 for each gain in per cent. The great gain made by Case T was caused by a clause in the specifications in regard to reversal of stress; in this case there was no reversal of stress in the upper chord members, while in Case B every member was reversed; the same thing was true for the diagonals and verticals with the possible exception of one or two members in each set. Therefore, if the design were made under a smaller factor than 80 per cent., Case T would not be as large.

In view of the above, the writers claim it is only fair to state that where three-hinged spandrel arches are to be designed, Case T will be the cheapest and most advantageous.

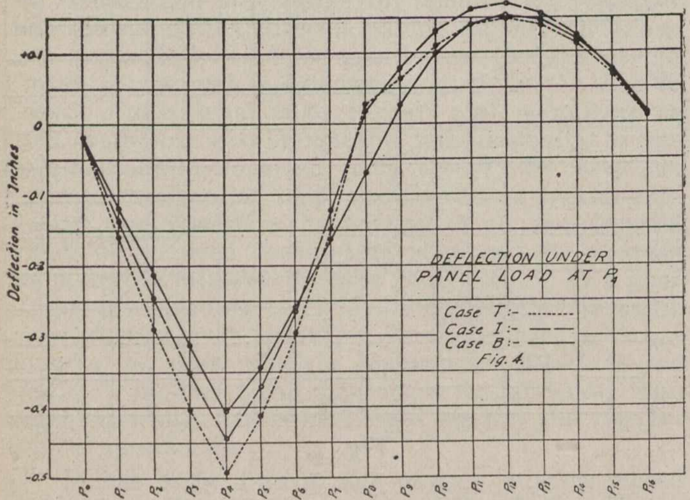


Fig. 4.

Influence of Deflections.—The stresses, sections, etc., which were found in the first study for a comparison of weights were used by R. W. Parkhurst to find deflections. Four cases were considered, those under dead load, full live load, and concentrated panel loads at the quarter points and middle of the span respectively. The stresses due to the load P_4 were computed directly by the analytic method and those for the loads P_8 and P_{12} were found by the graphic method. From the stresses and given sections, the elongations were computed and deflection diagrams drawn. The results are given in the form of curves in Figs. 2, 3, 4 and 5.

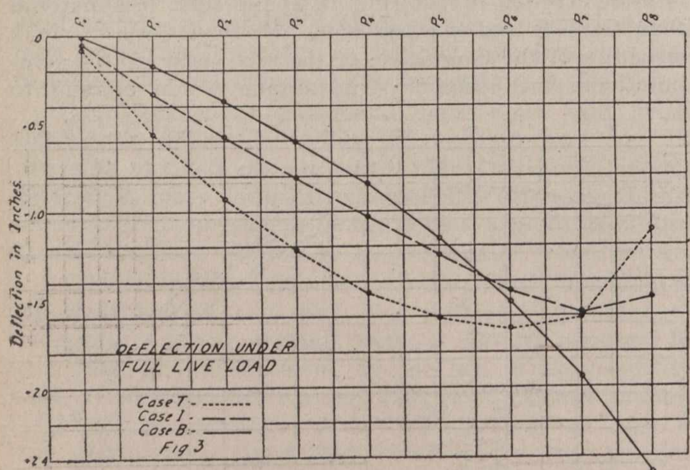


Fig. 5.

Conclusions.—Examination of the curves indicate that under dead load and full live load, Case B possesses greater stiffness at the quarter point than the other types, but deflects considerably at the centre. Case I shows good average values of deflection, while Case T shows great deflection at the quarter point, buf comparatively little at the centre.

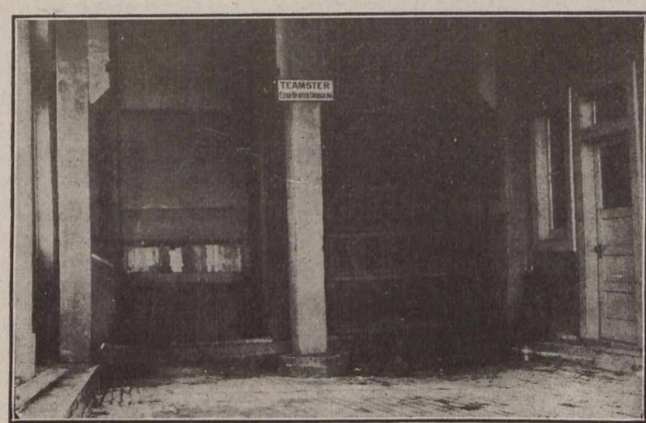
For the load at the quarter point, Case B shows greater stiffness throughout, while Case T allows considerable deflection under the load. but about the same amount of upward deflection for the unloaded half of the span, as Case B. Case I indicates average deflection under the load, but greater upward deflection. Both Cases I and T remain about stationary at the centre.

Under the load P_8, Case B shows greater deflection than the other types, Case I again ranking as intermediate.

From the preceding results, Case T shows considerable deflection in all cases except under the load P_8, while Case I seems to give the best average throughout. Case B is not considered on account of its comparatively great weight as shown above. For extremely light loads, Case T might be employed on account of cheapness of material, but since Case I is but 3.4 per cent. heavier, the extra cost should not be great, and where the loads are larger, and where additional stiffness is required, the latter arrangement might well be employed to advantage.

ELECTRICITY IN INCINERATOR OPERATION.

THE municipal incinerator just accepted by the city of Pasadena, Cal., has at least one electrical feature that is unique, a system of four elevators which receive the loads from the dump wagons at the ground level. These cars are of iron and each has a capacity of about two tons. The dump wagons shoot their loads directly into the elevator car, which is set with its upper edge flush with the floor, as shown in Fig. 1. The car is then hoisted by electrical power to the top story of the building, running between upright tracks. At the top these tracks are curved in such a way as to turn the car almost upside down, leaving it at an angle



Electric Elevator for Receiving Garbage.

of 45° with its mouth below the base and flat against the furnace door. The door is then hoisted, also by electrical power and the contents of the elevator are shot into the furnace. Besides this automatic dumping device, the fan which adds to the draft of the 152-foot smokestack is electrically operated, while, of course, the lighting is electrical. Current is obtained from the municipal lighting plant, which is only 500 feet distant. It is charged against the incinerator account, and costs about 25 cents a day for operating the hoist and about $1.50 a day for operating the blower. In constructing the incinerator, it was planned to install a large boiler on the top floor to take advantage of the resulting heat from the furnaces and generate steam power to be delivered to the city lighting plant, but this plan has not been carried out as yet. The incinerator is a reinforced concrete structure, costing $46,000 and has a capacity of 30 tons of garbage a day. It is of the Fredsmith type, and depends largely upon the dry rubbish delivered as a fuel to destroy the wet refuse.

GARBAGE DISPOSAL COSTS.

THE question of garbage disposal has reached an acute stage in Chicago, the dispute between the city and the former contractor for disposal and the inability of the city authorities to decide upon new methods of disposal have continued until the old contract expired and have resulted in a refusal of the contractor to continue pending settlement of the question, so that now the property owners are begged to dispose of the garbage as nearly as possible upon their own premises, and the city is treating with chemicals what it must collect before dumping it in abandoned clay holes.

The discussion has resulted in the presentation of much information of greater or less value upon the comparative values and costs of various methods of disposal. The following information, from "Municipal Engineering," is derived from comparative estimates made by M. de Ronore, of Paris, France:—

An assumption of 1,000 tons of material for disposal per day is made, of which, according to the Parisian average, 40 per cent. will be garbage and street sweepings and 60 per cent. dry, combustible rubbish. The average cost for incineration of wet garbage is said to be $2 a ton and that of incinerating a mixture of equal parts of dry waste and wet garbage over 50 cents a ton. The dry, combustible rubbish has a calorific value one-fifth that of good coal, while the mixture with garbage has no calorific value.

The mixed method of garbage disposal produces a fertilizer weighing about 20 per cent. of the garbage treated, this weight being about one-half that of the portion of the garbage reducible to fertilizer. The remaining three-fifths of the mixture is burned and produces electrical energy. The cost of a plant of twelve groups, each group with a capacity for 10 tons per hour, is estimated as follows:—

Land, 4.5 acres to be furnished by city, is not included:—

Buildings for the plant	$ 300,000
Offices and lodgings	60,000
The Mechanical Installations—	
Wagon-loading apparatus	$ 60,000
Sorting, crushing, etc., machinery	80,000
Producer system incinerators	220,000
Boilers	120,000
Electrical machinery	180,000
Expenses, loading, freight, etc., on construction materials and machinery	30,000
Paving about plant	50,000
Conduits and pipes for water and electricity	80,000
Side track	40,000
Miscellaneous	40,000
Contingencies	80,000
Total estimated cost of plant	$1,340,000

The cost of operation of this plant is not stated, but the amount of labor required is estimated at somewhat less than twice that required in an incinerator plant and about one-fifth that required for a reduction plant of about the same capacity.

The income from the plant using the mixed method is estimated as follows:—

The 400 tons of organic matter per day will produce 200 tons of fertilizer, or 73,000 tons per year. This is valued at $5 a ton, or $365,000 a year. The combustible waste is estimated to produce 50 k.w.-hr. of electricity per ton, and the 600 tons per day (219,000 tons per year) would produce 10,950,000 k.w.-hr. per year. Deducting 2,190,000 k.w.-hr. required about the plant leaves 8,760,000 k.w.-hr. for sale, which, at 2 cents, would produce $175,200. The total gross income from the plant for year would, therefore, be $530,200. In addition, there would be income from sale of rags, metals, etc., recovered.

These estimates may not fit American conditions exactly, but can be modified to fit them, and are certainly interesting. Even after deducting an expense of operation equal to that of running an incinerator plant with the addition of the greater labor cost in the mixed method plant at, say, $1 a ton incinerated, a handsome profit remains for the mixed method.

THE STRENGTH OF WIRE ROPES.

IT is frequently assumed that the stress causes in wires by bending them over pulleys is considerably lower when the wires are twisted in one or several strands than when single wires are tested. According to Bach, the former stress would be only three-eighths of the latter, and some authorities have based their regulations as to the strength of lift-ropes on the assumption of this coefficient. Bach was attacked in 1907 by Isaach-Sen, and the latter was supported by Bock, and last year by Wehage, who directed attention to the strains left in the ropes by the stranding. "Glück Auf," in a recent number, furnishes an interesting contribution to this question, though it is only based on preliminary experiments. Professor G. Benoit and Mr. Woernle are engaged on an investigation of the strength and durability of wire ropes, which they are conducting in the laboratory for hoisting-machinery of the Technical High School at Karlsruhe. This research will occupy them for some time to come; but as their experiments are pretty conclusive about the deleterious influences of twisting, they have published their preliminary results, which were reviewed lately in "Engineering," London.

The experiments in question were made with a patent cast-steel wire, 1 mm. in diameter, which they found had a strength of from 174 kg. to 180 kg. per sq. mm. (110 to 114 tons per sq. in.), the guaranteed strength being 160 kg. per sq. mm. The wire was tested as it was, and also twisted to a strand of seven wires, 3.1 mm. in diameter, there being six steel wires and a core of softer wire of a trength of about 86 kg. per sq. mm. This rope gave a strength of 805 kg.; the soft core was generally not broken in the test, because it stretched considerably. Three of these strands were then combined to a rope of 6.8 mm. in diameter, and five to a rope of 8.5 mm. in diameter; the latter cable was provided with a hemp core, the former not. The wires and ropes were applied to a pulley which was turned to and fro through an angle of about 90 deg. at the rate of 1,000 turns per hour; the bending and unbending of the wires thus took place always in the same direction; further experiments with alternating bending to different radii, etc., are now being made. A turn is understood to signify bending of the wire from the straight and back to the straight.

The size of wire under test was generally such that the stress amounted to 8 kg. per sq. mm. (5 tons per sq. in.). The pulleys were made of cast iron or zinc, and were used either in rough condition or filed or properly turned true. Though sufficient oil was always supplied, and there seemed to be no damage directly due to friction, the good finish of the pulleys had a very marked beneficial influence on the life of the ropes. The single wires stood 198,710 bends; the twisted strand had one or two wires broken after 44,800 and 47,190 bends. The pulley diameter was 175.4 mm.

A slightly larger pulley, 180.4 mm. in diameter, was then taken. On this single wires stood from 122,000 to 200,000 bends; the twisted strand became injured in three wires after 40,860 bends; the cable of three ropes had one wire broken after 22,860 bends, and was practically done for after 36,440 bends; the five-rope wire began to fail after 35,000 bends, and was given up after 40,000 bends.

The twisted ropes thus proved to be much less safe than the untwisted wires, and it occurred to the investigators that the twisting might in itself be responsible for the loss of strength. The wires, ropes, and cables (except the cable with a hemp core) were therefore annealed. In the case of the wires and wire bundles the annealing was fully carried through; the strength of the wire was thereby reduced to 93 kg. per sq. mm. In the case of the twisted ropes the heating was only carried so far, that the cut rope did not show any tendency to uncoil. The wires stood 47,700 bends, the twisted wires 37,000 and 42,000 bends, when the rope was destroyed; the cable of three ropes failed after 15,400 bends, and was quite destroyed by 21,850 bends. In other tests the untwisted wires stood from 47,700 to 70,900 bends; in the simple rope wires began to break after 50,850 bends, and the experiments had to be abandoned after 70,020 bends. That the cable of three ropes proved weakest may be due to special accidental features as to the relative dimensions of the parts. But the experiments thus demonstrate that the twisting leaves considerable strains in wire ropes, and especially on those made of high-class steels, which are chiefly used in mine haulage and winding.

LAYING SIDEWALKS IN WASHINGTON.

One-half of the cost of laying sidewalks in Washington, D.C., is assessed against the abutting property, and ordinarily the commissioners do not order sidewalk construction until they have received a petition from the owners of more than one-half of the frontage along a block. An exception is made, however, where a walk becomes dangerous, the commissioners order the work done in such cases without waiting for a petition. The law requires them to advertise for two weeks their intention to lay sidewalks and curbs, and, after a hearing, to order the work done when, in their opinion, it is necessary for the public safety, health, comfort and convenience.

During the last fiscal year, $225,000, according to The Municipal Journal, was expended in paving sidewalks abutting private property, and $7,000 in placing sidewalks and curbs around government reservations. The sidewalks were constructed of cement by contract. The alleys were paved with asphalt block or vitrified block by day labor, 23,422 square yards of vitrified block and 18,214 square yards of asphalt block, both on a gravel base, having been laid last year. Cement sidewalks were laid by contract for the following prices: Large jobs adjoining paved streets, 96 cents per square yard; large jobs adjoining unpaved streets and all small jobs, $1.20 a square yard. Contracts have been let for next year's sidewalk work at 92¾ cents and $1.16¾, respectively.

PRODUCER GAS FROM WOOD.

PRODUCER gas from wood was dealt with in a lecture recently delivered by G. E. Lygo before the Junior Institution of Engineers. It was only recently, he said, that gas plant makers had given serious attention to the subject of utilizing wood for use in gas producers. This was no doubt due to the higher price of a wood waste plant as compared with an anthracite plant and the low price of anthracite, but with the increased cost of coal, manufacturers having combustible waste materials of little value and difficult to dispose of were looking out for means of utilizing them, and the advent of reliable wood waste suction plants had given them an opportunity of effecting considerable economy. In these plants all kinds of wood, from sawdust to pieces 6 inches in diameter, cotton seeds, cocoanut shells, fiber, and dust, sugarcane, coffee husks, rice husks, spent tanning bark, rubber leaf waste, surface peat, etc., could be used.

Referring to the design of the plant, many points had to be taken into consideration which were not needed with an anthracite plant. The area of the generator was governed by the nature and size of the fuel, but, roughly speaking, it was 2½ times that required for coal fuel. The depth of the fuel depended upon its size and density. A deep fuel bed was necessary for large pieces of wood, else air passed through and ignited the gas in the top of the generator.

Small, dense fuel, such as sawdust and coffee husks, required a comparatively shallow bed, or its resistance would interfere with the working of the engine. The internal hopper must be designed to suit the fuel, some fuels falling evenly in the generator, while others formed a cone in the centre. The external hopper was of large capacity. A vaporizer was not required, as there was no necessity to keep down the temperature in the combustion zone, and the loss of hydrogen was more than made up by the volatile gases in the fuel.

The gas must be cooled and washed immediately it left the generator, else the heavy tar and dust in suspension would be deposited and choke the connecting piping. In Whitfield plants an anti-fluctuator was provided, which insured a steady flow of gas from the generator. The variation of pressure on the suction stroke rarely exceeded 1 inch of water, whereas 6 inches was not uncommon in plants without the anti-fluctuator.

The calorific value of wood when air-dried was approximately 6,000 B.t.u. per pound. The amount consumed per brake horse-power depended upon the moisture, which amounted to 10 to 20 per cent. in air-dried wood and 30 to 50 per cent. in fresh timber. Fuel which contained excess of moisture had to be dried until it did not exceed 60 per cent., otherwise it was difficult to make the generator fire burn evenly. The gas produced was of higher value than that from anthracite, namely, about 150 B.t.u. to 170 B.t.u. per cubic foot, as against 120 B.t.u. to 140 B.t.u. from the latter.

As regards cost of working, a test made with a Whitfield producer supplying an 84 brake-horse-power engine coupled to a 105 volts dynamo gave 540 B.t.u. for 2,072 pounds of fuel consumed, or 3.83 pounds of fuel per kilowatt. The test extended over 10 hours, and the fuel consisting of oak and elm sawdust and chips from working machinery, had no value for other purposes.

In one factory, before the installation of the producer, the same work had been done by a steam plant, when, in addition to the same amount of wood waste, upward of 500 tons of coal per annum were consumed.

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CONTENTS OF THIS ISSUE.

ADOPTION OF LIQUID FUEL FOR MARINE WORK IN BRITISH COLUMBIA.

That oil for fuel is speedily replacing coal on British Columbia harbor craft is a fact that is daily becoming more noticeable. Practically all the Government vessels plying in and out of Canadian ports on the Pacific Coast are using oil burners, or will be using them in another six months. The Marine Department was the first to make the change, while now the tug boats, dredges, snag-scows, tenders, etc., operated by the Department of Public Works in Vancouver and Victoria harbors, and on the Fraser River, are experiencing a similar conversion.

It has been found that this is a change for greater efficiency, and particularly so in the operation of dredging machinery. A reduced pay roll is noticeable in the stoke-hold of the vessels that have adopted the new system, about half the number of firemen being required as when coal was the steam producer.

Of course, the adoption of oil has advanced its price. At the present time, merchants are maintaining but a slight difference in the matter of cost between oil and coal. Nevertheless, the marine yards at Esquimalt are doing a rushing business in the conversion into oil burners of a veritable fleet of dredges, tugs, and the like.

Among those vessels upon which the oil system has been found to give excellent service are the Mastodon, operating around Vancouver; The Fruhling, keeping down the sand banks on the Fraser, and the Lobnitz, engaged in rock excavating in Vancouver harbor. The New Westminster stern-wheel suction dredge, King Edward, and the dipper dredge, Ajax, of Victoria, are among the craft at present at Esquimalt undergoing the installation of the oil system of steam generation.

FEDERAL CITY PLANNING COMMISSION.

Ottawa, like Washington, seems destined to become a purely parliamentary centre. The House and the Senate have never been enthusiastic about making the city a great manufacturing or industrial point. Early this month the Ottawa citizens voted by a decisive majority against the continuance of the publicity bureau that had been organized to attract industries. The civic body seems to be in sympathy with the desire of Parliament to make Ottawa a real Capital—a Federal City—a place of dignity, impressiveness and majesty.

In this work the chief factor will be the Federal City Planning Commission, recently appointed and as yet without permanent staff. This Commission is said to be contemplating improvements for Ottawa and surrounding district that will likely total fifty million dollars. The work will be spread over a period of twenty or twenty-five years, in all probability. Federal buildings, parks, railroads, streets, boulevards and the general view from Capital Hill will be within the scope of the Commission's activities. A Capital of a half million population will be planned for—the Commission will not be near-sighted.

The work includes Hull as well as Ottawa—on account of the importance of Hull as the background for the Capital. The Rideau Canal and the Ottawa, Rideau and Gatineau Rivers are also included—with the possibility of the Rideau River being diverted—and of a huge joint power scheme being developed. Great changes will gradually be wrought in a territory about sixteen miles square.

And may we add the hope that when—in 1934—the visitor to Ottawa has been duly impressed with the dignity and importance of Canada's Capital, that he will be able to obtain a drink of pure water—whether from the Gatineau Lakes or from a mechanical filtration plant.

FACTORY CONSTRUCTION IN REINFORCED CONCRETE.

There are two types of reinforced concrete factory construction—the one with outside bearing walls of concrete and few openings, the other, the skeleton type of construction, the walls being simply filling-in panels, built afterwards. It has been pointed out that the latter is the easier to build and the more economical, but that it calls for a consideration of the following points: The column and pilaster footings only need go down to a solid bearing, unless an excavated basement is required. Where there is a basement light walls reinforced horizontally from column to column, or vertically from the basement floor to the first floor, will retain the earth, the reactions being taken by the columns or by the basement and the first floor, while the walls may be reinforced to carry themselves from footing to footing, requiring no foundations of their own.

Where there is no basement the outside walls need only go far enough down to prevent frost working in under them, with possibly a shallow trench filled with cinders or gravel underneath. They can be reinforced to carry themselves from pier to pier and to support the walls above. By building the footings first, and carefully filling, settling, and levelling the earth and laying the floor on the ground, the shores to support the false work can be cut of even lengths, and there will be a good level surface to shore from. Columns and floors are built first, as in skeleton steel construction, and the outside panel walls are self-supporting, but not weight-bearing. They are built in between the pilasters entirely independently of the floors. They may be built at a later time, furnishing a convenient method of keeping the concrete gang busy while the concrete floors are setting or the wood forms are being shifted from floor to floor, or when the weather is too wet or too cold to permit of the laying of the more-important work of floor construction.

CANADIAN RAILWAY CONSTRUCTION IN 1913.

During the past year Canada's railway lines were extended by a total of over 3,000 miles of single track, not including some 450 miles of second tracking. Comparing this with 2,230 miles in 1912, 1,898 miles in 1911, 1,844 miles in 1910, 1,488 miles in 1909, and 1,250 miles in 1908, the mileage returns exhibit a rapid increase.

As will be noted by the table of statistics given on another page of this issue, the volume of construction work that has developed during the year greatly exceeds the figures of any previous year. There are over 3,000 miles in various stages of actual construction at the present time.

The Canadian Northern added over 1,275 miles to its trackage; 206 miles in British Columbia, 500 miles in Alberta, Saskatchewan and Manitoba, and about 580 miles in Quebec and Ontario. The Canadian Pacific laid nearly 600 miles and 200 miles on its Western and Eastern lines respectively. The Grand Trunk Pacific laid 480 miles in the West. The Transcontinental added 90 miles.

In addition to the above figures the Canadian Pacific built over 450 miles of second track, approximately two-thirds of which belongs to the main line west of Winnipeg.

Comparing Canada's 1913 railway building with other American countries, it is noticeable that the total single track construction of United States' railways during the same period of time is approximately equal to Canadian development, the difference being remarkably small. In 1912, the difference was approximately 750 miles. During both years, however, the United States railways have exceeded Canada's second track additions by about 900 miles.

The tempestuous conditions that have obtained in Mexico during the past year have made an enormous difference in the returns for construction activities in that country. During last year only 38 miles of line were completed as against 210 miles in 1912, and 350 miles in 1911.

LETTERS TO THE EDITOR.

Re the Proposed New Water Supply for Ottawa.

Sir,—It is a most extraordinary thing to find to-day in this country a plan to go to distant lakes at an expenditure of about $8,000,000 for the water supply of a city of the size and location of Ottawa, seriously recommended. Still more extraordinary is it to find it seriously considered.

There are to-day dozens of purification plants in this country and over the border in successful operation which, with some slight modifications to meet conditions of the Ottawa River water, would satisfactorily purify and render colorless the present Ottawa supply.

From the writer's experience with a 25,000,000-gallon filter plant, handling during part of each year the same water as obtains at Ottawa in the Ottawa River, he can say that $550,000 would erect and completely equip a 25,000,000-gallon filter plant of the highest efficiency at Ottawa which would give an absolutely satisfactory result using the Ottawa River water.

The above figure does not include the necessary connections with the city's present pumping plant as the cost of this would depend upon the relative location of the two plants.

But, as to the feasibility and absolute fitness of a modern so-called mechanical gravity filtration plant for the Ottawa water supply, there is not the least doubt.

Why throw away about $7,450,000 on the undertaking and then perhaps leave doubt as to the absolute safety of the supply?

F. H. PITCHER,
General Manager and Chief Engineer,
Montreal Water and Power Company.

Montreal, Jan. 6th, 1914.

* * * *

On Expert Opinions of the Fixed Carbon Test.

Sir,—The writer has followed with considerable interest the series of articles appearing in your publication on the fixed carbon question and, with your permission, will supplement his original contribution with the following remarks:—

Three of your contributors advocate a fixed carbon requirement in some form or other; yet their different views as to its application add more to the uncertainty of the subject than they accomplish as arguments for such a requirement.

Let us assume for the present that the test itself is a scientific determination which gives consistent results in the hands of skilled operators and that engineers are not interested in requirements which favor or discriminate against any special product or class of bituminous materials. With this hypothesis, let us review the statements of your fixed carbon advocates.

In your issue of November 13th, Mr. Pullar recommends that different fixed carbon limits be set for the various classes of asphalts in competition. This would, of course, be fair to all materials provided, of course, the engineer did not neglect any of the "classes" in his specification, but would it not be a step in a dangerous direction? Would not a manufacturer be justified in asking that different limits be set for other requirements such as ductility, susceptibility, etc., in order to suit the characteristics of his special product? At best this would involve complicated specifications likely to revert us to the practice of purchasing by "brand," the elimination of which is the very fundamental principle of the open scientific specification.

Furthermore, how would the method take care of mixtures of asphalts often used with much success, also fluxed materials. I have in mind the practice of some cities which base their requirements not on a crude or refined material but draw the specification for the finished asphalt cement ready for the paving mixture.

In Mr. Kirschbraun's discussion of December 4th, it is recommended that fixed carbon be limited by a purely arbitrary formula of which two essential factors are the fixed carbon and asphalt yield of the crude. As there is considerable variation in refining practice, how will the testing laboratory arrive at the yield of asphalt? Will they determine this by some laboratory test which may or may not represent manufacturing figures or will they look to the producer for this data? Without means of verification in either case, the inspecting chemist will be safer in basing judgment on the finished product alone.

Mr. Richardson, in the issue of December 18th, voices our indebtedness to Messrs. Pullar and Kirschbraun for their efforts in showing the direction of importance and interpretation of the fixed carbon requirement. How do these directions coincide with his recommendations as a result of so many years of experience? Both commentaries oppose any definite maximum limit but, to use his words "if the highest grade of material is desired to the exclusion of the cheaper residual pitches made from oils . . . . ," let us see what protection the city will secure by adopting Mr. Richardson's maximum limit of 15%.

In the sixth paragraph of Mr. Pullar's argument he mentions an asphalt showing 12.32 per cent. of fixed carbon which "was very poor and not suitable for bituminous work." Later on, in the twelfth paragraph we have, in striking contrast, the statement that "owing to peculiar characteristics of oils obtained from Mexico . . . a much higher fixed carbon is obtained . . . averaging between 14% and 18% for the well-known brands," and still further on we read that "Mexican oils will give satisfactory results despite comparatively high fixed carbon." I quite agree that the two gentlemen have done much to clear up the fixed carbon subject certainly in so far as a definite maximum limit is concerned.

The writer has not approached this subject in a commercial manner with the object of exploiting or condemning a brand or class of material, neither is he adverse to tests which show up inferiority but he has, independent of this discussion, found "fixed carbon" too empirical to be of any use as a refinery or inspecting test.

We may theorize as to "cracking," "over-heating" or other evidences of "improper refining," but if the test by which these defects are to be determined is a variable quantity then our structures of theory and argument become mere creations of sophistry. Take the fixed carbon reports of my sample No. 215, published in your issue of November 27th, 1913. From the variety of results submitted this sample could fall within several of Mr. Pullar's "classes"; would meet a like judgment by Mr. Kirschbraun's formula even knowing the fixed carbon and asphalt yield of the crude; also from the report of laboratory "B" it is within Mr. Richardson's flat limit of 15% though made from Mexican oil. Which of the six reports will one select?

To the writer it would seem that such chemical problems, which are at best uncertain, would be avoided by engineers until settled by the profession to which they more appropriately belong. If, however, the engineers wish a harmless excursion into the less familiar field, let them call for fixed carbon reports from several chemists on the same sample when, I feel sure, the absurdity of the situation will be realized.

LEROY M. LAW.

Baltimore, Md., Jan. 6, 1914.

* * * *

A Reply to Prof. Richardson re "Fixed Carbon."

Sir,—I notice in your issue of December 18th an article by Mr. Clifford Richardson—who, as he states, has had many years' experience in the determination of the fixed carbon test and other characteristics of native bitumen for the Barber Asphalt Paving Company—as to the desirability of maintaining said fixed carbon determination in specifications drawn for the purchase of asphalt.

Mr. Richardson argues:—

"If an engineer, in the light of service tests, prefers material which has been well proved to be satisfactory, rather than residual pitch which has been a failure or is of an experimental nature, he will properly introduce into his specifications a provision that no bitumen would be acceptable under them which contains more than 15% of fixed carbon."

I am surprised that Mr. Richardson should at last acknowledge the real purpose of the fixed carbon test, and advise that an engineer should introduce that test in his specifications, in order that only the expensive natural asphalts—Trinidad and Bermudez—shall be used, when a franker method would be to specify these materials by name.

The fixed carbon test, except for the purpose of making a market for the so-called "natural asphalts" above stated, is now abandoned, and all up-to-date municipalities by abandoning the same are obtaining the advantage of competition, and are securing asphalt that has been proven better and cheaper than the impure natural asphalts, which were used in the early days of the paving industry, simply because there were no others. Much impartial expert testimony has proven that the fixed carbon test has no value in determining the quality of asphalt or its fitness for paving material.

EDWARD SLADE.

Montreal, January 5th, 1914.

RAILWAY TUNNELLING.

By Leonard Goodday, C.E., M.E.,
Late of the British Admiralty.

THE following is not presented as being new to engineers, nor to contain formulæ or positive rules for constructing such works, as none can be laid down; no two tunnels ever being exactly alike in conditions affecting their construction. However, it is hoped that assistants to engineers and contractors will find the information of material practical value.

Tunnels should be sparingly used. They require great care and honest workmanship in construction, repairs being generally accompanied by great expense and delay to important traffic.

Light and Heavy Ground.—When a cutting attains 70 feet in depth it is generally advisable to tunnel. A cutting of this depth, for a double line of rails, with 27 feet width at formation, and 1½ to 1 slopes, contains about 1,027 cu. yd. of excavation per yard run. A tunnel in sound rock requiring no lining is a comparativelv simple piece of work, but those in unsound ground, requiring a lining, are sometimes very difficult and troublesome, especially where the height to the surface of ground over the tunnel is small.

As a fact, the greater the distance between the surface and the top of the tunnel, the lighter will be the ground generally, and therefore the greater the ease of construction. The reason is that when near the surface, mining operations cause the whole of the intervening ground to be disturbed. Cracks appear on the surface, and the whole weight of the earth has to be borne on the timber and lining. This condition of things is called "heavy ground." When the tunnelling is at a considerable depth, the disturbance caused by mining is arrested before it comes to the surface, and the weight is borne in the ground itself, i.e., "keyed in," while the ground is then called "light." Ground of 50 feet or less is likely to be heavy unless the geological formation is a strong one. When operations are commenced the strata and direction of "dip" should be studied so that the probable pressure may be ascertained. Sinking shafts and driving headings will assist in this work.

"Clay backs" often exist between the faults in rock, the stone having been upheaved and broken in enormous wedges, the sides forming a zigzag line vertical to the line of the tunnel. Water, having run between the sides of these wedges, has deposited clay which readily becomes slippery. The first length may be driven through one of these wedges, with its apex downwards. Two, perhaps three, lengths may be got in, and no great weight encountered, where the wedge is pointing downwards. If the wedge points upwards a great weight is suddenly brought upon the timber, because the rock, having its base cut away, slides down between its clay sides, and an increased thickness of lining will be necessary.

Clay is not a desirable substance to tunnel through, however sound it may appear. Water and air have a great effect upon it. The former renders it unstable and dislodges it from behind the timber, causing the latter to give, while the air makes it swell, crushing and breaking the timber.

In heavy tunnelling a thick lining is required, which cannot be determined upon until the work has been partially opened out. In light tunnelling the same process must also be gone through, but in a modified form.

Sinking Working Shafts.—The line once determined, it does not follow that shafts should be permanent, but one may well be so if the tunnel is over 700 yards long and if there will be much traffic. This shaft will be useful for steam clearance and ventilation purposes. If an engine passing through leaves the tunnel full of steam, the engine driver of the next entering train is unable to see his signals. The number of shafts to be sunk is regulated by length of tunnel and time allowed for its completion. Two faces should be worked from each shaft, and a length for each mined and lined complete for, say, five yards in every 20 shifts or 10 working days and nights. One shift would work from each shaft and one work to the shaft from a "break-up." Allowing 600 working shifts per year and a full gang for every two faces, 30 lengths would be mined and lined per annum in each direction, 15 being worked from "break-up" faces.

Suppose a tunnel of one mile length has to be completed in one year, how many shafts should be sunk?

One should be sunk at each end of the tunnel, and mining carried on from it in one direction only. In the other direction a heading may be driven, for expediting the excavation of the cuttings leading up to the tunnel, if heavy. The possible work to be done in both directions is 300 yards per annum, and in 1½ years 450 yards, leaving 1,310 yards to be worked from, say, 3 intermediate shafts. With this number it may be completed in the specified time. Sinking these 5 shafts and driving the headings will take at least 6 months. A "break-up" should not be commenced in a heading until the headings are all through, and centre line accurately fixed. It will be seen from this that for a tunnel 1 mile in length with 5 shafts 2 years is not too long. Shafts should be nearly equidistant from each other and the lowest points should not always be chosen for a site; for, though of less depth to sink, surface water and pumpage will naturally drain into it. Water is one of the greatest obstacles encountered in tunnel works. Also, the "bank" or top of shafts require raising above the surface, to get tipping room for the excavated material. Temporary shafts should be at least 9 ft. square and of timber, and sunk vertically. The settings of timber should be sound, 10 by 10-in., halved into one another at the angles, and plugged through the halving. They should be 4 ft. apart, centre to centre, the first one placed about surface level. Poling boards 1½ or 2 in. thick will not be too strong for lining, and if thicker ones are necessary short ends of planks will serve efficiently. The first setting having been let in sufficiently to keep steady, the poling boards are set up around the outside and excavation carried out inside and from under the "setting," the boards being hammered down and following the excavation. Temporary props are kept under the angles to prevent settling. When 4 ft. has been excavated another setting can be placed vertically under the first, taking care that the lower ends of the first set of poling boards are behind the second setting, and so on. If the ground around and close to shaft is level, the pit mouth should be built up a few feet, this giving height for a tip for excavation. A "jack roll" may now be set up over this shaft with 2 small skips attached on opposite sides of the winding barrel, so that as one is ascending the empty one is descending. For raising the excavation and water this will serve very well for depth of, say, 40 ft., but a winding engine or horse gin will eventually be required for handling the material.

While the above is going on, arrangements must be made for erecting a "poppet head" over the pit's mouth, and forming a gin ring with excavation already coming up. When a pit is over 25 yds. deep, a horse gin is too slow, and a winding engine is required. It should not be less than 14 h.p., and should be of the ordinary portable type—with its wheels removed and the engine resting on a solid bed. It should operate by belt, a friction winding gear, allowing continuous operation of the engine, and doing away with dangerous stopping and reversing. The skip descends by gravity and is regulated by a powerful foot-brake. A distinctive mark is often fastened to the steel wire rope to indicate, in passing over the pulley of the head gear, that the skip is at a certain distance from the bottom of the shaft; another mark near the other end of the rope indicating the skip's position when near the top. A bell should also be connected with top and bottom of shaft, and arranged to automatically ring when the skip is nearing top or bottom. When a skip is descending, the engine, although doing no work, should be kept running, and when a full skip is attached, and ready for lifting, the hanger-on at the bottom rings the bell, whereupon the brakesman simply throws the drum into gear with the engine. This does away with all jerking.

Suppose a lining is fixed at 2¼ ft. or 6 half brick courses, that the lengths proposed shall be 5 yards each, and that the bars necessary for supporting the ground while it is being mined and lined shall be 1½ ft. in diameter at the butt end. Suppose also that the weight upon the bars will cause them to sag or settle down 12 in. in depth before the arch is keyed (which is called the "drop"—the bar must be kept up to that extent to allow for such sagging) that the brickwork may be put in of full thickness under the sagged bar. It follows that the top side or "back" of bar must be 2½ ft. higher than the extrados, and that the under side or belly of the head-tree of the top heading must be kept higher up and above this to allow for the bar being placed under it. Drive the top heading, and on arriving at the two open ends of the tunnel, the passage through it will be found at a higher level than the formation of cutting, and a consequent lift of some 15 feet for all material passing in or out of the tunnel. Next decide upon the position of the "break-up" between the shafts. A "break-up" is formed by opening out and lining a length midway between two shafts, after the heading is driven between them. This creates at once

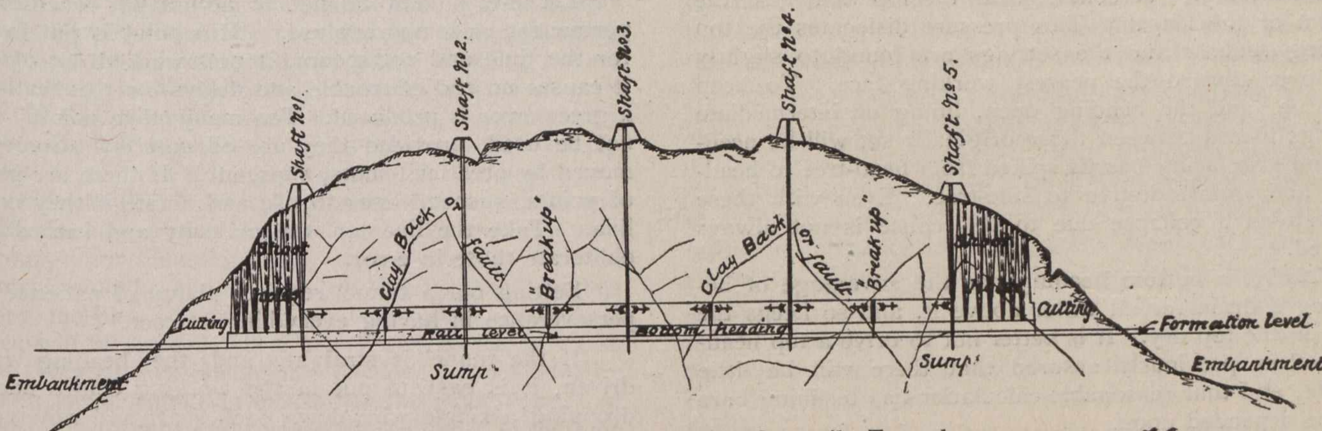

Fig. 1.—Longitudinal Section of a One-mile Tunnel.

Over the pit-mouth a bogie, or rolly, on 4 iron wheels about 12 or 15 in. in diameter and of a gauge equal to the full width of the shaft, should run on light rails laid back into a sort of dock sufficiently far to allow the rolly to be pushed back clear of the pit mouth. On the rolly are laid light rails at 2-ft. or 2-ft. 6-in. gauge on which are run small trollies. The full skip ascends clear of the mouth, the rolly with the trolly on it is pushed over the mouth and the skip then lowered upon the trolly. It is then disconnected from the winding rope and an empty skip attached. The rolly is then pushed back into the dock while the trolly and skip are run to the tip head.

At the bottom of the shaft a sump about 3 yds. deep is formed in the same way as the shaft, to collect water. A frame is built over it to form a cover to withstand stones, etc., which may fall down the shaft.

Headings.—A heading must be driven as a communication all through and between the working faces, and for accurately setting out a centre line below ground. Either a bottom heading at formation, or a top heading with its top a little higher than the extrados of the arch. A top heading must be made for each length, being necessary for lining in. It is necessary, also, for getting the "crownbar" into position. It is argued that, as the driving of this heading is most expensive in mining for the bars, why not drive right through at the outset. This is the only good argument in its favor as against a "bottom heading."

two new working faces, and facilitates the progress of mining and lining. It also permits more hands being employed, and the progress is thereby expedited. Each face is worked by a gang from each shaft. As the "break-up" is formed from a top heading whose bottom is about 15 feet above formation level all excavation below the bottom has to be raised into the heading and to the face working from the shaft; thence down that face to the shaft and up to the surface. Water must be dealt with in the same way until the faces from the shaft and "break-up" meet one another.

These are not the worst arguments against a top heading. Bearing in mind that a certain thickness of lining and that certain bars with a specified drop will be sufficient, the first length is commenced. Before about three are completed the calculations are found valueless, and that a lining 3 feet thick, bars 2 feet diameter with 18 in. drop are absolutely required to take the weight. Top of heading must be raised 1½ ft. or 2 ft. all through the tunnel despite the increased weight owing to the ground having been previously disturbed.

This raising in wet and heavy ground is exceedingly difficult, and will be found to cost fully 50% of the first cost of the heading. It may be claimed that the heading should be driven high enough at first. In this case if the first calculations are borne out this will entail all the more dry packing to be done between the brickwork and the roof of the mined ground. Again, in driving a top heading at the outset, considerably stronger timber would be used, as some of it must remain under strain for approximately half the time during construction of tunnel; whereas, the timber used for supporting a top heading driven for each length as it is worked, would only remain understrain about 10 days, and consequently need not be so strong.

If "drawing bars" are used, they are placed under the head-trees of the top heading, the head-trees forming the top timber of the settings placed in this heading to support the ground, until the crown bar is in position and staved, remaining there until the brickwork of the length is keyed. The bars are then drawn, and serve for the next length. The head-trees cannot be extradicted and are lost (which supports the contention that this timber should be no stronger than necessary). In a light tunnel no "drawing bars" are needed, what are called "taking out bar" being used instead. In this case the head-trees can come out with them.

There is always considerable face weight in a length, i.e., pressure tending to shove the face forward into the length, and often causing a good deal of movement, more, of course, near the top than near the bottom of the length. This pressure dislocates the top heading timbers and the settings are found to slightly lean over toward the nearest working face. To keep them up, and the heading open, lining or intermediate settings placed between those originally set will be needed, and also lacing boards spiked from head-tree to head-tree, and from side-tree to side-tree. Even with these precautions a collapse due to this cause is not always averted.

Lastly, a bottom heading has the advantage of becoming a drain for all water above it, and so keeps the work in the top dry. It is better not to drive a top heading unless previously assured that there will be little weight, and that reasonable calculations as to lining bars can be depended upon.

Setting Outlines; Driving Headings, and Fixing Position of Working Faces.—As it is more desirable to drive a bottom heading, let it be supposed that all shafts are sunk to formation level, with a good sump under each with, for example, one shaft at each end and three intermediate shafts placed nearly equidistant.

The centre line must be accurately set out on the surface and permanent points fixed upon it, one on each side of each shaft and on the tangent, if there is a curve. They must be clear of all workings and not disturbed. It is well to suspend two copper wires from the top of either of the shafts, set up the transit over one of the points and sight along to the other across the shaft. Fix two pieces of wood on some convenient part of the headgear transversely, one on each side of shaft, and as far apart as the width of shaft will allow, so that the wires may hang freely from them. Cut a notch in each board in the centre line and suspend the wires in the notches with a 10-lb. plumb-bob. These bobs should swing in a pail of muddy water, or oil, care being taken that the wires are clear of timber, etc. Their line can be transferred to some fixed point on either side of the shaft, or by stretching a fine cord across them, which will be sufficiently accurate for driving 10 yds. of heading on either side. When this length is done more accurate lines are required. The wires are again swung and the transit taken down the shaft. Then, with a plumb-bob attached to a fine line, sight on to the line of the wires as nearly as possible with the eye. Indicate in some clay, put on the ground for the purpose, where this bob drops; set the transit over this mark and sight onto the nearest of the two wires. To provide sufficient light for this work, a hole should be pierced in one of the trunnions and a mirror and reflector placed in it, then, by holding a candle to the end of this trunnion and looking through the eye-piece, the hairs will be visible. If electric lights are used this operation will be facilitated. A convenient method of sighting the suspended wire is to have a piece of white paper held behind it, with a light behind the paper. Set the transit on this wire, then have the wire gently moved to one side and the cross hairs should cut the other wire. If not, then the point in the clay is a little out and the transit must be gently shifted until the wires coincide exactly with the cross hairs of the telescope. The centre line being now on the instrument, an iron staple may be driven into the head-tree of the heading, in the setting next but two to the face. In it file a nick exactly on the centre line, and repeat this on the other side of the shaft, without moving the instrument. By hanging plumb lines in these nicks an accurate line is obtained which will serve for another 20 yards or more each way.

Let every point be put in accurately, whether for permanent or temporary use. If a point is put in just for the time and not secure, it gets worked out of line, it causes no end of trouble and delays for rectification if a great error is produced. Too many "line points" cannot be used, provided they are all correct. Some get moved by pressure, shots, slips, etc. If there are plenty of points, suspend lines to all, and notice if they are in line. Take the line of the majority and immediately obliterate those in error.

Setting out a tunnel requires plenty of patience and perseverance in having everything correct.

If the tunnel is straight, and the heading fairly driven, it is easy to put in the working centre line by dropping a single wire down each shaft and ranging in the true centre already found.

Now set the transit at one shaft, and sight to the next; then reverse to that in the opposite direction. If the cross hairs do not cut on to both wires correctly, halve the error at both shafts, shift the instrument forward, and try the next shaft. Thus all five wires may be put into line. Now fix points in the heading between the wires at, say, every half chain, by driving a staple into the head-tree and filing a nick in it. Verify these points now and then by going over this process, as sometimes the mining operations cause the ground above to settle, shifting these points, and throwing the wires out of line.

The best way of giving lines on a curve is to drop one line down the shaft exactly in the centre, set the instrument in the line tangent to the curve, and let a second wire be dropped in this tangent line. Then, with the transit produce the tangent into the heading, placing points at every half chain, measuring the offsets to the curve from each point. Drive in staples and file nicks as before. Make a neat plan of this, setting out the points plainly, and give it to the foreman-miner, who should be a man of intelligence and able to use it. If a point is blown out by a shot, the foreman can help himself if he understands the plan.

Always set out centre lines with an instrument, for if carried out by rule of thumb, errors which, although not serious, are annoying, creep in, causing loss of time. Besides, if headings driven from opposite faces do not meet well, a "jink" is caused in the line of temporary rails afterwards laid, and wagons will be continually running off at the jink. In a close heading the derailment of a full wagon is a great source of annoyance.

A bench mark having been established at the bottom of each shaft, by downward measurement, a "brob" should be driven into the timber at rail level and painted. If the tunnel is on a gradient a board about 15 ft. long, with one edge level and the other cut to the inclination of the gradient, should be fixed into position. The gauger can then level from the B.M. until sufficient heading is driven to allow of another B.M. being put in. The bottom of the heading should be kept about 1 ft. above the intended formation level. The size of heading, clear of all timber, should be enough to permit the free passage of a tip-wagon, for as soon as the headings are through, a line of rails will be carefully laid through the entire length of tunnel and connected with the cuttings at either end, and also with the outside works. This road will be commenced for bringing in heavy bars, timbers, bricks, mortar, etc., and for removing the excavated material.

A heading 7½ ft. high and 8½ ft. wide at the underside of the head-tree, and between the side-trees, will be sufficient. While sinking the shafts, etc., a good supply of small timber should be provided for this, averaging from 6 to 10 in. diameter, and also plenty of poling boards from 1 to 1½ in. thick, 6 to 9 in. wide, and about 4 ft. long. If the ground is loose and shaky, a great many such boards will be needed, and as the immediate supply of them direct from a lumber merchant is uncertain, and being without them is as bad as being without bricks and mortar, the lumber should be on hand. To ensure a constant supply for a 1-mile tunnel, at least two circular saw benches should be set up, and driven by the engines working the pits. A great many foot blocks, cleats and wedges will be needed, all of which can be cut by these saws. This sawing should not be done by piece-work; it causes much trouble in measuring the work done, and the miners will constantly come up the shaft for boards and wedges, which will not have yet been measured. The result is that the sawyer's word must be taken as to quantity, as it is impossible for the timekeeper or foreman to be always at hand.

The side-trees, when in position in the heading, should each have a 6-in. "sprag," i.e., they should lean inward to that extent. Hence, if the distance between them at the underside head-tree is 8½ ft. at the bottom of the heading, it should be 9½ ft.

Every side-tree in a heavy tunnel should be set upon a foot block of hard wood about 15 in. square and 3 or 4 in. thick, and placed about 6 in. below the proposed floor of the heading. This gives a wider bearing surface, and prevents the tree from sinking or cutting into the ground. Sometimes the floor of the heading will commence to rise or spring up, owing to the pressure above. If there are any signs of this, every second setting of side-trees, and sometimes every setting, should be placed upon a good square timber or sill, running across the heading from one side to the other, below temporary rail level. The weight on the side-trees keeps the ends of this timber in place, and the weight of the road and passing wagons keeps the middle from rising. Two side-trees and one head-tree, and a sill, when necessary, form a "setting." These settings should be placed at least at yard intervals, the poling board reaching from one head-tree to the other, and overlaping 3 or 4 inches. In soft ground these boards must be placed close together throughout.

[NOTE—Mr. Goodday's article will be continued in an early issue.—Editor.]

CALCIUM CARBONATE IN WATER.

AMONG the papers discussed at the annual meeting in London of the Institution of Water Engineers of Great Britain was one by William T. Burgess, F.I.C., on the solubility of carbonate of lime and its bearing on certain processes for the treatment of water supplies. The following constitutes the dominant features of his address:—

The majority of water engineers have to deal with more or less calcareous supplies, and there are two groups in particular who have to study the matter practically. On the one hand, many are in charge of works where the supplies are softened, the operation being mainly the removal of carbonate of lime, and, on the other hand, there is another group concerned in hardening their soft supplies, usually by the addition of carbonate of lime, with the object of neutralizing acidity and preventing possible action on lead.

The fact that carbonate of lime is such a common ingredient in hard waters was doubtless one of the reasons which made the late Dr. Clark select it as the representative substance in his well-known soap test, and to express degrees of hardness in terms of that carbonate.

Some of the best water-bearing formations are entirely, or largely, composed of carbonate of lime—e.g., chalk, oolite, limestone, etc.—but although waters derived from such areas are always hard from the presence of carbonate, the carbonate of lime itself is really a very insoluble substance in pure water. Many chemists have estimated the solubility, the results varying from 1 to 3 parts per 100,000 of pure water. Careful experiments by the author, using water as pure and free from carbonic acid as possible, have shown that 100,000 parts will only dissolve 1.5 parts of the carbonate.

Carbonic acid is the natural agent which accounts for the carbonate of lime in our water supplies, but it is incorrect to assume that the rain water which falls on calcareous areas carries sufficient carbonic acid to effect the solution. Rain water seldom contains more than 1 part of carbonic acid per 100,000, a proportion quite insignificant towards effecting the solution of such amounts as are found in chalk-derived waters. A much larger amount of carbonic acid comes from the decomposition and oxidation of the organic matters in the soil on which the rain falls, and the examination of water from a percolation gauge having a depth of only 3 ft. of soil-covered chalk has shown that such water may already contain half as much carbonate of lime as is found in water from many deep wells in the chalk. The carbonic acid holds the carbonate in solution as bicarbonate. Water saturated with carbonic acid at ordinary pressure and temperature will take up as much as 100 parts of carbonate per 100,000 of water, but nothing like this amount is ever found in public water supplies. From numerous analyses the author has made it appear that the proportion of carbonate of lime in calcareous waters is seldom over 30 parts per 100,000. (Carbonate of magnesia is much more soluble than carbonate of lime, both in pure water and in water saturated, or partly saturated, with carbonic acid.)

The elimination of the carbonic acid, which is loosely combined with the carbonate to form bicarbonate, will cause the separation of the carbonate of lime. Thus, boiling for some time drives the carbonic acid out of the water, and results in the deposition of the carbonate; the water is softer after the operation, and on this account the hardness due to carbonate of lime is described as "temporary" to distinguish it from the "permanent" hardness caused by other soap-destroying ingredients which are not affected by boiling. Heat does not drive off the carbonic acid readily, and consequently the boiling process is neither economical nor efficient for softening water. It is not sufficient to heat the water to boiling-point; it must actually boil, and often five minutes' brisk boiling will only reduce the hardness due to carbonate of lime by about 50 per cent. In the ordinary domestic hot-water services to baths, the water rarely reaches boiling-point owing to the circulation, and but little of the carbonate of lime, such as is found in our London supplies, gets deposited in the boilers and pipes. The author has frequently tested the hottest water from bath taps, and seldom found the carbonate hardness reduced by more than one-eighth of the total. If the carbonic acid left the water easily on the application of heat, and thus caused a better separation of the carbonate, the users of calcareous waters would be obliged to have their hot-water boilers and pipes cleaned out at much shorter intervals of time than they do now.

After long storage calcareous waters tend to lose carbonic acid, particularly in open reservoirs where vegetation flourishes. In such cases the carbonic acid is utilized by the plants, and deposition of a certain amount of carbonate takes place, with corresponding reduction of the hardness of the water.

Most calcareous waters contain a little carbonic acid in excess of that in hypothetical combination as bicarbonate, and this excess can be partly removed by agitation with air; the removal of the excess causes a slight separation of the carbonate of lime. This is not a practical method of softening, but the author found that the carbonate hardness of a chalk well water was reduced from 23.5 to 10.4 parts per 100,000 in the course of some months by storage and occasional agitation with air. (Vegetation had no part in the reduction, as the water under experiment was kept in the dark.)

Many chemical substances may be added to water to combine with the carbonic acid, and thus reduce the precipitation of the carbonate; but of all these lime is the cheapest and best.

Removal of Carbonate of Lime from Water by Clark's Lime Process.—As is well known, the hard water is mixed with a proportion of lime water just sufficient to react with the carbonic acid which holds the carbonate of lime in solution; the carbonate originally present, together with that formed by the combination of the carbonic acid and lime, becomes insoluble, and falls out as a fine crystalline precipitate, leaving the water softened. This general statement requires a little qualification, for, as already explained, carbonate of lime has a slight degree of solubility in water entirely free from carbonic acid, the amount being 1.5 parts per 100,000. From this it follows that it is impossible to reduce the carbonate of lime hardness in the most favorable water below that figure—at any rate, by any simple lime process. (As an illustration of how close works practice may approach the theoretical limit, it is, perhaps, worth mentioning that the author has on many occasions found the carbonate hardness of softened water below 2 parts per 100,000.) The carbonate of lime is either allowed to settle down in large subsidence tanks, or, after giving a short time for the reaction, it is filtered off.

Considering, first of all, the method of subsidence in tanks, it is a matter of common observation that the time taken before the water clears is by no means constant. Sometimes the water is bright in six or seven hours, while at other times twelve to fourteen hours elapse before the precipitated carbonate has properly subsided. The reason for this variation, where the conditions are apparently identical, is obscure; but it probably depends on the way in which the lime water and hard water happen to mix, intimate mixture and good agitation favoring the formation of a fine precipitate. Although twelve to fourteen hours is usually sufficient for the subsidence and the production of brilliant water, there is a great advantage in allowing many hours more, for the water, although clear, is often in the condition of a super-saturated solution of carbonate of lime, and from such a solution the excess carbonate, while it does not form a further precipitate, will crystallize out slowly on the sides of the tanks, or, if sent out too soon, in the mains. One of the objects of this communication is to call attention to the fact that the full separation of the carbonate is often a matter which requires a much longer time than is usually supposed. The subject is best illustrated by the results of a few laboratory experiments. In the tests the waters were treated with the proper proportions of lime water to effect good softening, and after the precipitate had subsided, the perfectly clear top was carefully syphoned off at intervals and examined for total carbonate in solution.

A. **Chalk-derived Water.**—Total carbonate hardness in untreated water 22.5 parts per 100,000.

	Total carbonate found in clear water after softening.			
Time of subsidence ..	10 hrs.	25 hrs.	49 hrs	73 hrs.
Total carbonate in clear softened water in parts per 100,000	4.8	3.9	3.6	3.6

B. **Calcareous Water.**—Total carbonate hardness in untreated water 17.6 parts per 100,000.

	Total carbonate found in clear water after softening.			
Time of subsidence..	10 hrs.	22 hrs.	46 hrs.	70 hrs.
Total carbonate in clear softened water in parts per 100,000	3.1	2.8	2.4	2.1

C. **Chalk-derived Water.**—Total carbonate hardness in untreated water 22.4 parts per 100,000.

	Total carbonate found in clear water after softening.				
Time of subsidence..	12 hrs.	24 hrs.	36 hrs.	60 hrs.	84 hrs.
Total carbonate in clear softened water in parts per 100,000	3.7	3.3	3.2	3.0	3.0

A consideration of the above results is quite sufficient to show that even when perfectly clear water is drawn off and pumped to distribution there is more than a probability of the crystallization or separation of a small amount of carbonate of lime in the mains. The longer the time the water is stored before distribution the less the probability of deposit.

The author has made many experiments to ascertain the effect of partial softening in the delayed separation of the carbonate. In the tests recorded below the waters were treated with the correct proportion of lime water (A), and also with smaller proportions to effect less softening (B, C and D).

Calcareous Water.—Total carbonate hardness in untreated water 17.2 parts per 100,000.

Time of subsidence.	Total carbonate found in clear softened water, in parts per 100,000. A	B	C	D
10 hours	3.3	3.8	4.1	5.8
22 "	3.0	3.6	4.0	5.8
34 "	2.6	3.3	3.7	5.7
46 "	2.5	3.1	3.6	5.6
70 "	2.3	3.0	3.5	5.5
94 "	2.1	2.9	3.4	5.4
Total carbonate which separated out after the water had first cleared	1.2	0.9	0.7	0.4

Chalk-derived Water.—Total carbonate hardness in untreated water 20.1 parts per 100,000.

Time of subsidence.	Total carbonate found in clear softened water, in parts per 100,000. A	B	C	D
11 hours	3.4	4.2	5.8	7.2
23 "	3.2	4.0	5.6	7.2
47 "	3.1	3.8	5.4	7.1
71 "	2.2	3.7	5.4	7.1
Total carbonate which separated out after the water had first cleared	0.5	0.5	0.4	0.1

Only one conclusion can be drawn from the above figures, and that is that the more perfectly the water is softened the greater is the necessity for giving it a long period for subsidence, and it is the irony of fate that at works where the most efficient softening is aimed at the probability of deposition of carbonate in the mains is greater than at places where partial softening is the rule.

With regard to softening works where filtration is relied on for the removal of the carbonate, the question arises: Is such filtered water more or less liable to form an after deposition of carbonate than the clear water obtained from subsidence water? In the author's opinion the liability is certainly not greater.

At what may be termed filtration works the hard water and the lime water are intimately mixed in their proper proportions at once; this favors the formation of a very fine precipitate, a large portion of which remains suspended in the water during the short time, usually about one hour, until the turbid water reaches the filtering cloths, where the precipitate is arrested. The surfaces and pores of the cloths soon become coated with the fine crystalline precipitate, and in making its way through what is practically a porous surface of carbonate of lime crystals, the water tends to part with any excess carbonate which it really holds in solution. This action compensates for the shorter time taken in the filtration process, and thus places the clear products of the two systems nearly level with regard to their liability to form deposits in mains and pipes.

After all, it may be argued that at well-conducted works not much trouble arises from deposition in the mains, but the author desires to point out that it may be practically prevented by simple chemical means. The addition of a very small amount of acid to the treated water from either subsidence or filtration works will entirely prevent the after-separation of the carbonate, and carbonic acid is probably the cheapest and easiest applied. In this connection members may be reminded that in the rapid softening process of Archbutt and Deeley,* in order to prevent the after-separation of carbonates from the treated water, the water leaving the softening tanks is "carbonated" by the introduction of air charged with carbonic acid drawn from the chimney of a coke stove. The author has reasons for thinking that the application of such "carbonating" at ordinary softening works would often be useful to engineers. The introduction of a very small proportion of carbonic acid, even as little as 0.3 part per 100,000, is sufficient to check or stop the after-deposition of carbonate, and at places where smokeless, or nearly smokeless fuel is burned there is no reason why the ordinary flue gases, after washing, etc., should not be utilized. At subsidence works the possible output might be greatly increased, for with slight "carbonating" the water could be drawn off and pumped to distribution, or reservoirs, immediately after it had first cleared. In carrying out any experiments it would be necessary to proceed with caution, because the introduction of too much carbonic acid might lead to other troubles, but the author believes that the complete control of carbonating plant would not be difficult.

Introduction of Carbonate of Lime Into Water: Hardening.—The amount of carbonate that a soft moorland water is capable of taking into solution is strictly limited, and is governed by the degree of acidity, which can, of course, be determined by analysis. Moorland waters always contain some free carbonic acid, and often, in addition, other acid or acids. If the free acid is simply carbonic acid, any proportion less than 0.7 part per 100,000 will only affect the solution of 1.5 parts of carbonate of lime, the same proportion as pure water will take up. With greater amounts of carbonic acid the proportion capable of passing into solution is exactly that which corresponds to the possible formation of bicarbonate; thus a soft water containing 1 part of carbonic acid will dissolve 2.4 parts of carbonate of lime. When the water contains, in addition, another free acid, this will determine the solution of a certain amount of carbonate and liberate a corresponding quantity of carbonic acid, which can take up a further proportion in the form of bicarbonate. Knowing the amount of acidity in the water, the chemist can therefore calculate the maximum proportion of carbonate which is possible of solution.

The carbonate is usually applied to soft waters in the form of powdered chalk, or whitening, and if the question of cost could be neglected, theory suggests that the material should be in the finest possible state of division. The fine particles expose a great surface to the water, and have the advantage of remaining in suspension for a long time. This is not altogether a matter of theory, for the author's laboratory experiments with carbonate in various states of division show the advantage of using very fine particles.

The president was kind enough to furnish the author with samples of untreated and treated waters from the Wakefield Corporation works. Members who visited the works this year will remember that the water is treated with both carbonate of lime and lime water in order to harden it slightly. The author treated the raw water with a considerable excess of pure carbonate of lime for two days in a closed vessel, and afterwards estimated the amount which had passed into solution. The result was 3.4 parts of carbonate per 100,000, and this figure was exactly the proportion which was indicated as possible

*Journal of the Society of Chemical Industry for 1891, page 511.

from the analytical determinations of the original acidity of the water. The author found that combined carbonate and lime treatment employed at the works had increased the hardness by the equivalent of 2.5 parts per 100,000, and that the process had been so carefully adjusted that the product contained neither free lime nor free carbonic acid. The above notes show what can be done in the way of hardening by carbonate of lime, and indicate the limits of the process.

"Excess Lime" Method of Sterilizing Water.—In the Eighth Research Report to the Metropolitan Water Board, Dr. Houston gave an account of experiments demonstrating the sterilization of raw Thames water by treating it with lime considerably in excess of that necessary to effect softening alone. The treatment of soft waters was also dealt with, and in the Ninth Report (April, 1913), this matter is followed up and notes are given showing the practical sterilization of a "very soft, peaty river water" by small quantities of lime. Experiments are quoted showing that purposely polluted water could be brought to a reasonable standard of bacteriological purity by the germicidal action of 2 parts, or even 1 part, of lime per 100,000 of water, the former proportion being effective in forty-eight hours and the latter in 144 hours. With regard to the free lime left in the water, Dr. Houston states that, if thought desirable, it could be removed by the addition of bicarbonate of soda. Dr. Houston recognizes that there would be cheaper methods of neutralizing the excess of lime "although none perhaps so apparently free from any disadvantages."

The expense of the suggested treatment will be better understood when it is explained that any free lime left in solution will require to neutralize it exactly three times its weight of bicarbonate of soda, and as this substance costs about £6 a ton, or roughly six times as much as lime, it follows that the expense of neutralizing will be 18 times the cost of the free lime left in the water after sterilization. However, neglecting the question of cost, the method may be examined from the chemical point of view. The reaction which takes place is—

$$Ca(HO)_2 + 2NaHCO_3 = CaCO_3 + Na_2CO_3 + 2H_2O.$$

Hydrate of lime + Bicarbonate of soda = Carbonate of lime + Carbonate of soda + Water

The solubility of the carbonate of lime formed has an important practical bearing on the possibilities of the process, and the matter is best illustrated by giving the results of some experiments. Samples of pure water, free from carbonic acid, were treated respectively with lime in the proportions of 1, 2 and 3 parts per 100,000, and were afterwards "neutralized" with 3, 6 and 9 parts of bicarbonate of soda. In the case of the experiment with 1 part of lime + 3 parts of bicarbonate of soda no precipitation or separation of carbonate of lime occurred, even after keeping the samples three days; but in the other two cases the precipitate formed slowly, and then settled down in about twelve hours, leaving the water perfectly clear and brilliant; but these waters were left in the condition of supersaturated solutions, from which the carbonate slowly crystallized out on the sides of the containing vessels, and were thus in states favorable for making deposits in mains. Further, it was found that the waters which were originally treated with 2 and 3 parts of lime, and neutralized with bicarbonate of soda, deposited, in the secondary separation, larger amounts of carbonate than might have been expected after allowing for the known solubility in pure water. The experiments were repeated several times, but always with the same results, and further tests established the fact that carbonate of lime is really more insoluble in water containing a little carbonate of soda than it is in pure water. (Thus, in water containing 3 parts of carbonate of soda, carbonate of lime is only soluble to the extent of 0.27 part per 100,000, as against 1.5 parts per 100,000 in pure water.)

It is fairly evident that if the free lime to be neutralized by bicarbonate of soda in the "excess lime" process exceeded 1 part per 100,000, special precautions would have to be taken, otherwise considerable deposits of carbonate of lime might readily occur in the mains.

THE CONSTRUCTION OF A REINFORCED CONCRETE RESERVOIR.

The accompanying illustrations and the following data respecting the design and construction of a reservoir and coagulation plant for the Anheuser-Busch Brewery at St. Louis, Mo., are from a paper to be read by Mr. Ed. Flad, M. Am. Soc. C.E., at the February 4th, 1914, meeting of the society. His paper includes a short explanation of the waterworks system of the company, the system having a capacity of over six million gallons per day, this water supply coming from the Mississippi River through two 20-in. cast iron intake pipes, being syphoned into intake wells, pumped into settling tanks, and clarified by chemical coagulant and rapid filtration before distribution. The chemical treatment was adopted in 1901, several years before the City of St. Louis itself began the treatment of water supply.

The low-service pumps of the system are in a brick pit, 30 ft. in diameter and 40 ft. deep. There are three centrifugal pumps having a combined capacity of 13,000,000 gal. per day, and one triplex, direct-acting pump having a capacity of 2,000,000 gal. per day.

There are two steel settling tanks, 75 ft. in diameter and 28 ft. high, one circular concrete reservoir approximately 150 ft. in diameter and 30 ft. deep, and one rectangular covered reservoir having a capacity of about 1,000,000 gal.

The filter plant comprises six Jewell filters, circular in plan, each 16 ft. in diameter, and three Reisert filters, recently completed, which are rectangular in plan, each being approximately 34 by 15 ft.

Chemical Treatment.—The water is settled by adding sulphate of aluminum (alum) and lime. A special three-story reinforced concrete building is provided for storing and preparing the chemicals.

The hopper for storing the lime is 36 by 9 by 14 ft. high, and has a capacity of 90 tons. It is placed in a pit so that it can be filled directly by shoveling from the cars. An electric elevator conveys to the third floor the hand-cars containing the lime or alum.

The alum is dissolved in a concrete tank, and is fed to the water by gravity. This alum tank has three rectangular divisions, each 10 by 7 by 5 ft. deep. Each division is charged with from 500 to 2,000 lb. of alum which is dissolved in water. It requires from 2 to 5 hours to dissolve one charge. The lime is slacked in iron tanks on the third floor. These tanks are rectangular, 12 by 12 by 7 ft. deep, with sloping bottoms. A false perforated bottom is provided at a depth of 30 in., on which the lime is placed and partly submerged in 1 ft. of water. After slaking, which requires about 1 hour, the attendant stirs the mixture, which passes readily through the perforated bottom and into the lime tanks below. This milk of lime is stored in three vertical cylindrical iron tanks, 12 ft. in diameter and 18 ft. high, with conical bottoms. One charge of lime consists of from 2,000 to 6,000 lb., and the tank holds about 10,000 gal. of water, giving a 2½ to 7% solution. In these tanks the milk of lime is kept agitated by compressed air admitted at the bottom through a small perforated pipe.

The flow of both the alum and lime solutions is regulated by standardized orifices operating under fixed heads. The milk of lime is pumped into the supply pipe or settling tanks by a centrifugal pump. As a general rule, the lime is added to the water as it enters the first settling tank, and the alum as the water enters the second settling tank.

Consumption.—The maximum consumption of water during the summer is about 6,000,000 gal. per day, for which at times, 1,600 lb. of alum and 5,600 lb. of lime are required, or about 2 grains of alum and 7 grains of lime per gallon of water.

The water is used for boiler purposes, for condensing and cooling in connection with the ice machine, as well as for washing barrels, bottles, etc. It is not used in brewing beer, partly because of the natural prejudice against water taken from the river below the outlet of the large city sewers.

Reinforced Concrete Reservoir.—Reverting now to the reinforced concrete reservoir completed: After due consideration of the various possibilities as to the shape and location of the reservoir, a circular shape was decided on, and the diameter was made as large as the available space permitted. The elevation of the top was fixed by that of the water in the old settling tanks, which operate in series with the new reservoir; and the bottom was placed sufficiently low to pass below the fill of cinders and rubbish, and rest on the river silt.

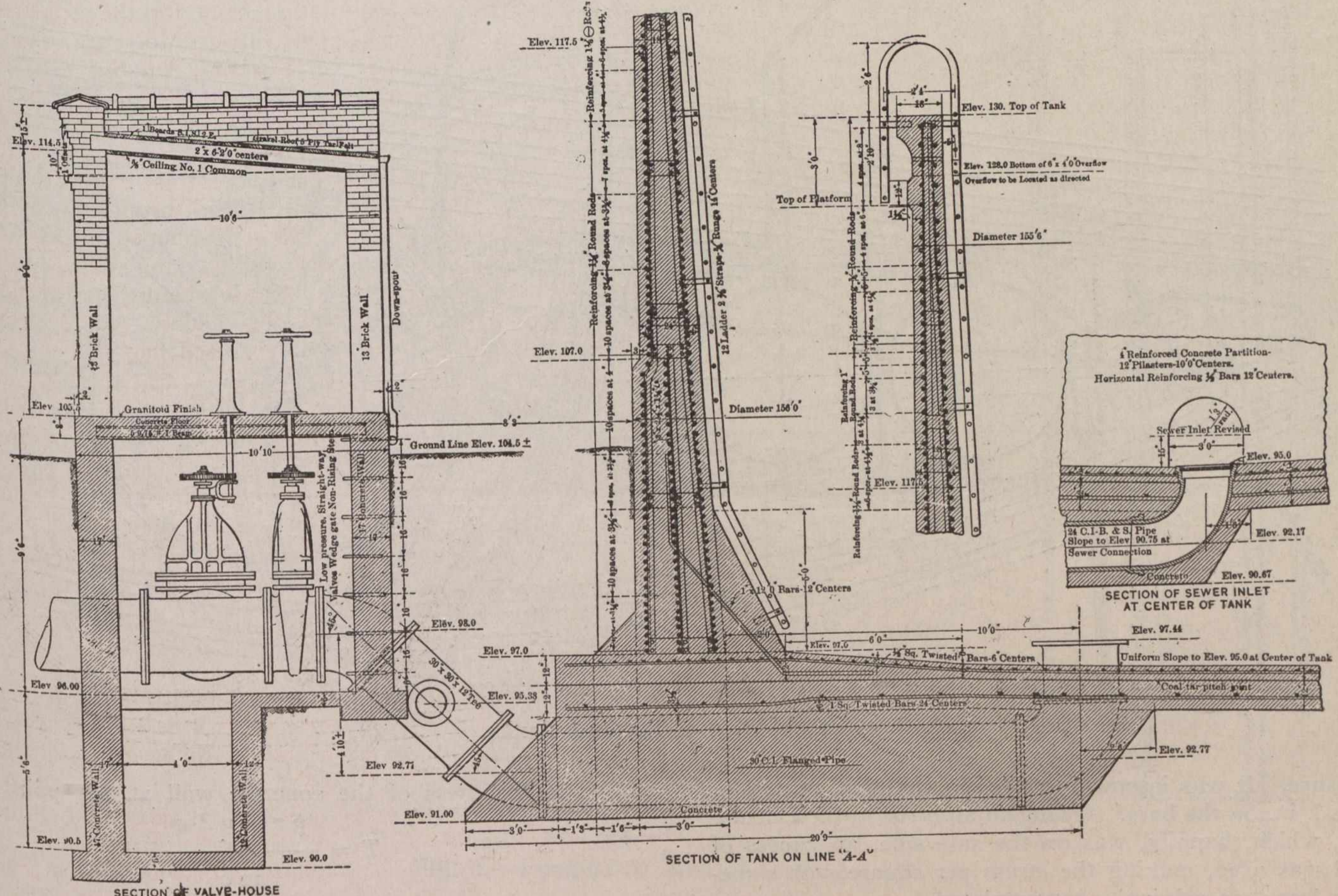

Fig. 1.—Section Details of Settling Tank.

Estimates were made of the comparative cost of a steel tank and a reinforced concrete reservoir. Exclusive of the foundations, pipes, gate-house, and accessories common to both designs, the steel tank was estimated to cost $32,000, and the reinforced concrete reservoir, $30,800. The reinforced concrete reservoir was supposed to have some advantage, being a more permanent form of construction and not requiring painting, and perhaps a desire to follow the latest fashion had a minor influence; at all events it was decided to use reinforced concrete.

The reservoir has vertical sides and is 153 ft. 6 in. in diameter at the top and 35 ft. deep at the centre. The side-wall extends 25 ft. 6 in. above the ground. The capacity is approximately 4,250,000 gal. There is a central partition, consisting of a 4-in. reinforced concrete wall, with buttresses, which starts at one side of the reservoir and passes diametrically across to within 14 ft. of the other side. The object of this partition is to make the entering water circulate around the reservoir before reaching the outlet. The diameter of the intake and outlet pipes is 30 in., and that of the waste pipe 24 in. The outlet pipe has a float and a hinged joint, so that water is always taken from near the surface. The valves controlling the flow are in a gate-chamber outside the reservoir.

Foundation.—The foundation is a 12-in. layer of concrete resting directly on the river silt and reinforced, in two directions at right angles, with 1 in. square bars, 2 ft. from centre to centre, making ½% reinforcement each way.

On top of this foundation rests the bottom of the reservoir, which is 6 in. thick and reinforced in a manner similar to the foundation, except that the bars are ½ in. square and 6 in. from centre to centre.

The top of the foundation was coated with a thin layer of coal-tar, the object being to provide for expansion and contraction of the bottom independent of the foundation.

Side-Walls.—The thickness of the concrete side-walls is 7 ft. 6 in. at the base, 2 ft. 5 in. at the ground line, and 12 in. at the top. The pressure of the water is carried by hoop tension, the side-walls being reinforced circumferentially with corrugated round bars, under the assumption that the concrete carries no tension. There are three lines of 1¼-in. round bars at the bottom and two lines of ¾-in. round bars at the top, the sections and spacing being varied from bottom to top to correspond with the pressure. It was intended to allow a stress of 15,000 lb. per sq. in. on the bars. Owing to an error in the calculation, which, happily, was on the safe side, an excess of steel was used, making the stress per square inch somewhat less than originally contemplated.

The thickness of the concrete wall was fixed at each point so that the concrete would not be stressed more than 290 lb. per sq. in., under the assumption that no vertical cracks would develop under this tension. Assuming the modulus of elasticity of steel at 30,000,000 and concrete at 3,000,000, the actual maximum stress, if no vertical cracks develop, will be 2,900 lb. per sq. in. of steel and 290 lb. sq. in. of concrete.

The following formulas were used in determining the dimensions and spacing of the steel bars and the thickness of the concrete:

A = Area of bars, in square inches per foot of height of wall;
D = Vertical distance, in inches, between two layers of bars at the point selected;
T = Thickness of concrete wall, in inches, at the point selected;
p = Water pressure, in pounds per square foot, at the point selected;
r = Radius of reservoir, in feet;
s = Stress in steel, in pounds per square inch, allowed, under the assumption that the concrete carries no tension;
c = Stress in concrete, in pounds per square inch, allowed;
a = Area, in square inches, of steel in each layer.

The area of the bars required per foot of height of wall is

$$A = \frac{p\,r}{s}$$

The vertical distance between the layers of bars is

$$D = \frac{12\,a}{A}$$

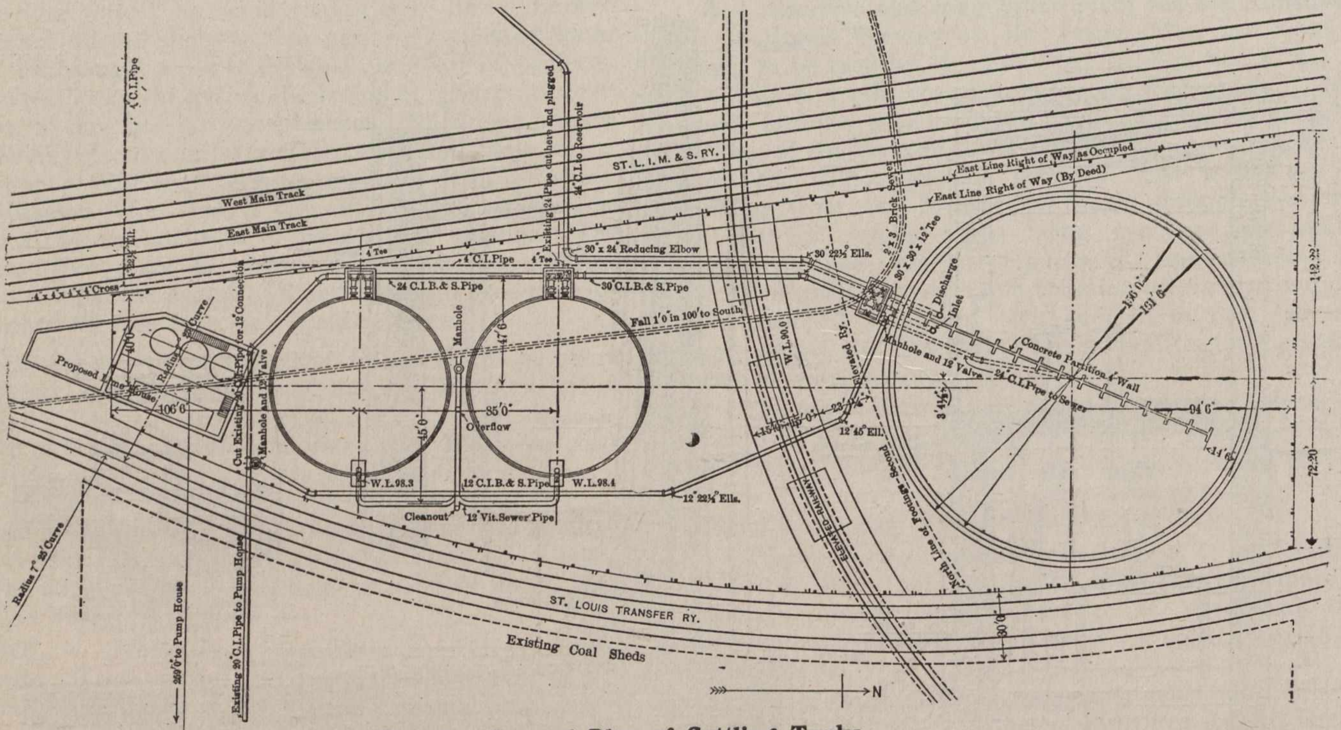

Fig. 2.—General Plan of Settling Tanks.

The thickness of the concrete wall at any point is

$$T = \frac{p\,r - 9\,A\,c}{12\,c}$$

The reinforcing bars were held in position by angle-iron frames, the sides of the angles being punched accurately with small holes for the insertion of the wires which tied the bars to the frame.

Splices.—The circumferential bars were in lengths of from 50 to 55 ft., and were spliced by lapping forty diameters and attaching two Crosby clips at each lap. The strength of this joint was tested by making up sample joints and pulling the bars in a testing machine. Two tests were made on each diameter of bar used, one with bars and clips without any mortar, and the other with the same joint embedded in cement mortar of the same mixture as that used for the walls of the reservoir.

The tests showed that the presence of the mortar did not add materially to the strength of the joint, as it failed either by slippage or by breaking off the clip. The first sign of slippage occurred at a stress of from 18,000 to

35,000 lb., and the ultimate strength of the joint varied from 25,000 to 75,00 lb.

Table I. gives the results of the tests.

Table I.—Tests of Bars.

Kind of bar.	Began slipping.	Broke at.
*1¼-in. round bar, plain......	....	75,600 lb.
† " " " with mortar	....	40,500 lb.
1⅛-in. round bar plain	37,400 lb.	....
" " " with mortar	32,200 lb.	32,200 lb.
1-in. round bar, plain	35,000 lb.	....
" " " with mortar..	25,600 lb.	25,600 lb.
¾-in. round bar, plain	27,300 lb.	30,000 lb.
" " " with mortar..	26,400 lb.	26,400 lb.

*1-in. slip at 18,000 lb.
†Mortar broke at 29,000 lb.

Forms.—The forms for the side-walls were built up of ⅞-in. tongued and grooved flooring, with 2 by 6-in. vertical studs about 2 ft. from centre to centre. The studs on the opposite sides were fastened together with iron wire passing through the forms. These wires were approximately 2 ft. apart horizontally and 4 ft. vertically.

The concrete was placed in the forms with a tower and dump cars running on a circular track.

Arrangements for Washing Out.—The bottom of the tank slopes toward the centre from all sides, the 24-in. waste outlet being at the centre. The slope toward the centre could not readily be made more than 2 in 75, being limited by the depth of the sewer which was already built. In order to facilitate the removal of sediment, therefore, the bottom was laid out in a series of star-shaped mounds, as indicated in Fig. 3, each mound draining into a shallow gutter. Hose connections furnish water under pressure for cleaning out.

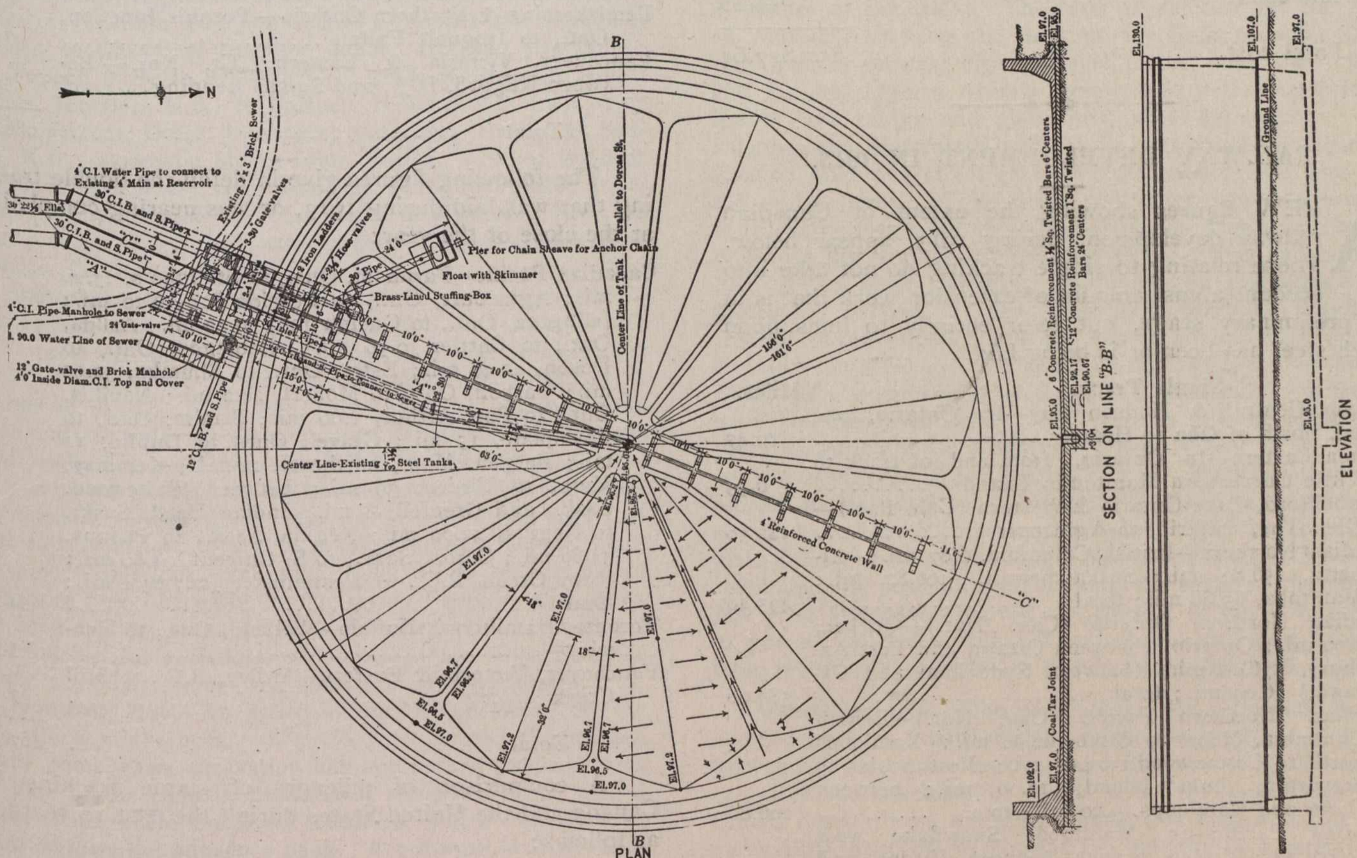

Fig. 3.—Foundation Plan of Settling Tank.

At the junction of the side-wall with the bottom, knee-bars are provided to reinforce this connection. These bars are 1 in. square and 12 in. from centre to centre.

Mixture.—The concrete in the walls and bottom is a mixture of 1 part cement, 1½ parts sand, and 3 parts limestone screenings run through a ¾-in. mesh screen, with the addition of 10½ lb. of Shamrock waterproofing to each barrel of cement. The composition of this water proofing is approximately as follows:—

Silica	60.0	per cent.
Alumina	15.0	" "
Lime	6.5	" "
Oxide of iron	1.5	" "
Combined water	10.0	" "
Gelatinous material	7.0	" "

This material cost 73½ cents per bbl. of cement.

Bond in Concrete.—Special precaution was taken to obtain a good bond between the successive layers of concrete. The old surfaces were scrubbed with brushes and a stream of water. In addition, 6-in. strips of corrugated plates, about 1/16 in. thick, were placed vertically in the joints. In spite of these precautions, when the reservoir was filled, small leaks developed along most of the joints, and efflorescence was quite extensive. These leaks appear to be closing up gradually, and it is probable that in the course of time they will disappear entirely. The intention, however, is to empty the reservoir and treat the joints with some waterproofing compound.

After completion the outside of the wall was rubbed with carborundum blocks and brushed with cement mortar.

The reservoir was designed by the writer's company, and was built under contract by the Fruin-Colnon Contracting Company. The cost, including pipes and accessories, was approximately $52,000. The unit prices and total cost of the various classes of work were as follows:

Excavation, 7,801 cu. yd. at $0.80	$ 6,240.80
Plain concrete, Class A, 58.2 cu. yd. at $4.00	232.80
Plain concrete, Class B, 281.3 cu. yd. at $5.00	1,406.50
Reinforced concrete, 12-in. base, 759 cu. yd. at $5.00	3,795.00
Reinforced concrete, 6-in. bottom, 473.1 cu. yd. at $8.00	3,784.80
Reinforced concrete side-walls, 1,178.3 cu. yd. at $11.00	12,961.30
Reinforced concrete partition, 120 cu. yd. at $20	2,400.00
Reinforcing bars, 582,668 lb. at 1¾ cents	10,196.69
Clips for reinforcing bars	650.00
Ladders and angle-iron supports	852.16
Cast-iron pipes and valves	7,283.00
Gate-house	1,200.00
Miscellaneous	844.59
Total cost	$51,847.64

RAILWAY DEVELOPMENT IN 1913.

A FEW figures showing the extent of Canadian railway development during 1913, appear below. Those relating to single tracking do not take into account a vast amount of extension work that is in the preliminary stage, but refer entirely to lines upon which steel has been or is being laid.

Single Track.	Miles.
Algoma Central & Hudson Bay—In Ontario, one mi. north of Oba to Hearst	49.47
Algoma Eastern—In Ontario, from end of track to Little Current on Manitoulin Island	6.55
Campbellford, Lake Ontario & Western (Can. Pac.)—Glen Tay, Ontario, to Agincourt	182.30
Canadian Northern—British Columbia, 6.07 mi.; Alberta, 256.89 mi.; Saskatchewan, 166.82 mi.; Manitoba, 65.66 mi.; total	495.40
Canadian Northern Ontario (Can. Nor.)—Quebec, 8.00 mi.; Ontario, (between Ottawa and Port Arthur), 536.00 mi.; (between Sydenham and Ottawa,) 36.00 mi.; total	580.00
Canadian Northern Pacific (Can. Nor.)—British Columbia, Hope to Cisco, 62.00 mi.; Kamloops north to Cottonwood, 123.00 mi.; Westminster to Steveston, Lulu Island, 12.00 mi.; between Cisco and Kamloops, 9.00 mi.; total	206.00
Canadian Pacific—In Manitoba, Snowflake west 10.00 mi.; Virden branch between Virden and McAuley, 23.00 mi.; between Boissevain and Lauder, 28.80 mi.; in Saskatchewan, Estevan northwest, 54.5 mi.; Kerrobert northeast, 36.10 mi.; between Swift Current, Sask., and Bassano, Alta., 60.8 mi.; between Weyburn, Sask., and Stirling, Alta., 162.00 mi.; in Alberta, Suffield southwest, 32.3 mi.; between Gleichen and Shepard, 25.00 mi.; on Alberta Central branch, 40.20 mi.; Lacombe east, 12.00 mi.; in British Columbia, on Kootenay Central 63.5 mi.; on Whitewater-Kaslo line, 16.00 mi.; total	574.20
Esquimalt & Nanaimo (Can. Pac.)—McBride Junction to Little Jualienne river	8.50
Fredericton & Grand Lake Coal & Railway (Can. Pac.)—In New Brunswick, Mile 11 to Mile 23	12.00
Grand Trunk Pacific—In Saskatchewan, Cut Knife branch, 30.00 mi.; Biggar-Calgary branch, 67.00 mi.; Prince Albert branch, 8.00 mi.; Moose Jaw northwest branch, 52.00 mi.; Regina Boundary branch, 48.00 mi.; in Alberta, Tofield-Calgary branch, 36.00 mi.; in British Columbia, west of Yellowhead, 122.00 mi.; and east of Prince Rupert, 117.00 mi.; total	480.00
Hudson Bay—Between The Pas, Man., and Port Nelson	100.00
Intercolonial—Georges river, N.S., to Sydney mines	8.80
Interprovincial & James Bay Ry. (Can. Pac.)—Lumsden's Mills, Que., north	10.00
Kettle Valley—In British Columbia	80.00
National Transcontinental Railway—In province of Quebec, 88.26 mi.; Manitoba, 2.22 mi.; total	90.48
Pacific Great Eastern—Between North Vancouver, B.C., and Dundrave, 4.50 mi.; between Newport, B.C., and Cheakamus, 13.50 mi.; total	18.00
Quebec Central—St. Sabine, Dorchester county, to St. Camille, Bellechase county	5.00
Reid Newfoundland Co.—In Newfoundland, Trepassey branch, Biscay Bay to Trepassey, 5.00 mi.; Carbonear to Bay-de-Verde, 53.00 mi.; Goobies to Black River, 15.00 mi.; extension Heart's Content branch to Heart's Content, 1.00 mi.; total	74.00
St. John & Quebec—Between Centerville, N.B., and Gagetown	90.00
Sydney & Louisburg—Waterford Lake, N.S., to Victoria Mines, 1.00 mi.; Morien Junction to Morien Village, 2.00 mi.; total	3.00
Temiskaming & Northern Ontario— Porquis Junction, Ont., to Iroquois Falls	7.25
Vancouver, Victoria & Eastern (Gt. Nor.)—Between Kilgard, B.C., and Sumas Landing	5.05
Total	3,086.00

The following figures give in detail the double tracking that was laid during 1913, or was nearing completion at the close of the year:

Canadian Pacific—Farnham, Que., to St. John's, 13.20 mi.; Agincourt, Ont., to Leaside Jct., 6.20 mi.; Islington, Ont., to Guelph Jct., 29.20 mi.; Azilda, Ont., to Cartier, 9.50 mi.; Hemegos, Ont., to Devon, 12.40 mi.; Ester, Ont., to Shumka, 32.90 mi.; Tarpon, Ont., to Mobert, 20.2 mi.; Navilus, Ont., to Port Arthur, 1.60 mi.; Semlin, Ont., to Paysiplate, 15 mi.; Gravel, Ont., to Dublin, 11 mi.; Bergen, Man., northeast, 20 mi.; Kemmay, Man., to Virden, 39 mi.; between Whitewood, Sask., and Grenfell, 8 mi.; Indian Head, Sask., to Regina, 42.00 mi.; Regina, Sask., to Pasqua, 34.80 mi.; Caron, Sask., to S. Current, 94.80 mi.; Ruby Creek, B.C., to Westminster Jct., 64.4 mi.; total	454.30
Toronto, Hamilton & Buffalo—Welland, Ont., to Fenwick	5.91
Vancouver, Victoria & Eastern—Ardley, B.C., to Still Creek	7.12
Total	467.33

A comparison of mileages of single tracking in Canada and the United States during the past 10 years is as follows:

Year.	Canada.	United States.
1904	316	3,832
1905	1,181	4,388
1906	1,007	5,623
1907	976	5,212
1908	1,249	3,214
1909	1,488	3,748
1910	1,844	4,122
1911	1,898	3,066
1912	2,232	2,997
1913	3,086	3.071

The Panama Railroad is now engaged in building a concrete sea wall along the front of the fill between the fire station at Cristobal and the end of the mole for the new piers, to be used by the buildings of the steamship companies. The total length of the wall will be about 350 ft. It is composed of concrete cubes, one yard on a side, which are laid in brick style and faced above the water line with a wall of concrete one foot thick.

COAST TO COAST.

Paris, Ont.—Hydro-Electric light was used in Paris for the first time on January 8th.

Nelson, B.C.—The plant of the Nelson Coke and Gas Company is being operated by the municipality.

Vegreville, Alta.—The main street of Vegreville is now illuminated by the town's recently discovered utility, natural gas.

Belleville, Ont.—The new branch of the C.P.R., known as the Campbellford, Lake Ontario, and Western Railway, which will connect Toronto and other Lake Ontario points with Belleville, will be opened for traffic shortly. The line runs between Agincourt and Glen Tay, a distance of 182 miles.

Montreal, Que.—The Intercolonial Railway of Canada has under way the following new construction: Nelson, N.B., to Derby Junction, diversion of line, 2.67 miles; St. Romuald, Quebec, to Chaudiere Junction, double tracking 3.75 miles; Pt. Tupper, N.S., to Sydney, N.S., grade revision, 91 miles; Oxford Junction, N.S., to Painsec, N.B., double track, 73.7 miles; Halifax Ocean terminals; passenger station at Sussex, N.B.; automatic blocks from Halifax, N.S., to Windsor Junction, 13.9 miles; from Moncton, N.B., to Painsec Junction, 7.2 miles; from St. John, N.B., to Hampton, N.B., 22 miles; and a line from North Sydney, N.S., to a point near Leitches Creek, 4.3 miles.

Vancouver, B.C.—Some indications that a regular service will be commenced next year on the P.G.E. railway north of Newport are an order recently placed by the company for 150 steel freight cars to be delivered at Newport during the months of March, April, and May next; the possible issue of an interim order by the provincial department of railways at an early date permitting the P.G.E. to operate trains over the completed portions of the line north of Newport in order to transport settlers going into the interior; the fact that steel rails sufficient to lay 30 miles of track are now being delivered at Newport, and that these are to be distributed along the new line early in the spring; and, finally, track has been laid 13 miles north of Newport and grading has been finished for a distance of more than 150 miles from the Pacific terminals, being well advanced beyond Lillooet.

Montreal, Que.—To guard against a subsidence of the soil, which might occur when the frost leaves the ground at any point where excavation has approached too closely to the base of the conduit, carloads of stone are being hauled to the place where the break occurred, which will be used to strengthen the aqueduct bank. A cribwork will be built along the conduit at a distance of a few feet; and after the crib has been filled with stone, earth will be used to fill in the space between the aqueduct and the crib. This work will form part of the programme for protection against water famine in case of emergency. The tapping of the Lachine Canal, the work for which is announced in the *Construction News* columns of this journal, is another phase of the same scheme.

Victoria, B.C.—Though about $2,000,000 of the sum of $9,682,600, appropriated last year by the Provincial Public Works Department, has not yet been spent, the amount of new construction which has been undertaken in public buildings, roads, bridges, wharves, etc., has exceeded greatly that of any previous year in the history of the Provincial Government. The mileage of new roads constructed amounts to over 700, and the mileage on which the department has been engaged, including the work done in repairs and improvements, amounts to over 12,000. Prominent among these roads is the Banff-Windermere highway, on which 16 miles of ungravelled roadbed have been constructed during the past season; and on the transprovincial road, at least 20 miles have been constructed through an abnormally difficult section. At Strathcona Park, under the supervision of Col. Thompson, Messrs. Casey & Lewis have cleared and grubbed 7.25 miles, which is now ready to grade, and have also built a permanent bridge. A large number of important bridges, from time to time noted in *The Canadian Engineer*, have been completed and a number are under construction at the present time. In view of the fact, however, that a large number of buildings cared for by last year's appropriations have not been completed, and also of the general relaxation in building activity, it is probable that the amount to be devoted to public works during the coming year will not approach the record sum expended last year.

Moose Jaw, Sask.—The total expenditure for the work carried out by the civic engineering department in 1913 was $378,059.13. Out of an estimated expenditure of $32,000 for the city engineer's office, when an expenditure of $18,761.26 was deducted for salaries and purchase, there was left a balance of $13,238.74. The total expenditure for the year on contracts covering the work on the high pressure dam, 11th Avenue subway, high pressure mains, sidewalks, curbs and gutters, Algoma Avenue sewer and water extensions, River Park bridge and abutments, was $212,86[illegible].12. The only bridges constructed were those over the high pressure dam on Manitoba street east, and the park bridge over the Moose Jaw river. In the sewer and water extension branch of the department, the annual report shows that 2,139 houses in Moose Jaw are now connected with the mains and sewers. Also in the street cleaning department, an expenditure of $9,600 has sufficed to keep in excellent condition a certain area of paved streets; and the report advises that by increasing the area, the cost could be lowered proportionately. Finally, the water supply of the city, it is stated, is in better condition than ever before; and there is now no danger of shortage during the dry months. There is kept in the reservoirs a supply of 37,990,000 gallons, to which another 30,000,000 gallons can be added at will. Realizing that steps should be taken to increase the water supply to keep pace with the increasing population, the commissioners have investigated the possibility of utilizing the water in the Moose Jaw river and have obtained the consent of the government to proceed with the work. They recommend, therefore, in their report, that particulars as to cost be obtained.

Vancouver, B.C.—The park scheme for some time being planned by park board officials, under the guidance of Park Superintendent W. S. Rawlings and Engineer A. S. Wootten, embraces the development of the point of land in the old C.P.R. hotel site as a picnic ground, the laying out of the parks adjacent to the beach, and the building of a permanent bath-house. The park officials believe that the whole improvement could be completed for $150,000, an equal amount of this being expended each year for the next four years. The plan provides also for the construction of a pier, which would cost probably $75,000; but it is not proposed to build this until necessary. A rearrangement of the car tracks of the C.P.R. operated by the B.C. Electric Railway will have to be made; and several new streets will also be laid out and approaches will be boulevarded so as to form a chain from Stanley Park and English Bay across the proposed Burrard Street bridge and out to Point Grey. The seawall will be constructed of rock taken from the foreshore, to be used as a facing, and to be backed by concrete. Towards the north end, it will be about 4 feet in height, increasing to about 10 feet towards the west; and, at intervals, there will be shelters and steps down to the beach. It will provide for a 40-foot promenade of asphalt with ornamental lighting standards every 75 feet along the edge. The bath-house will be built on the pivotal point of the scheme, equidistant from each end of the promenade; and will harmonize with the construction of the embankment. The seawall is expected to cost about $30,000 and the bath-house an equal amount. Many changes and improvements are planned for the grounds surrounding the beach, chief among which is a fill of 2 to 4 ft. that will be necessary immediately east of the northern portion of the beach. Though the board will be forced to forego much of the work this year, it will commence the clearing up of the west end of the beach, now strewn with rocks, and the piling up of those rocks preparatory to building the seawall at that section. The filling of the other section of the park, now undeveloped, must be proceeded with before permanent improvements can be performed there; and for this work, the park board expects to get the sand to be dredged out of English Bay in connection with the harbor work in False Creek.

PERSONAL.

JAMES E. WILSON, superintendent of the sewer department for the city of Calgary, has recently tendered his resignation. Mr. Wilson is leaving the city's service to engage in private contracting.

E. A. CLEVELAND, of Cleveland & Cameron, Vancouver, B.C., gave an illustrated lecture on the Panama Canal to the Vancouver Branch of the Canadian Society of Civil Engineers at its meeting on January 7th.

C. E. CARTWRIGHT and C. M. ODELL addressed the Mining Section of the Canadian Society of Civil Engineers at its meeting in Montreal on the 8th instant. The former's paper was entitled, "The Outlook for Mining in British Columbia." Mr. Odell spoke on "Initial Proceedings in Opening Up a Coal Mine."

JOHN R. FREEMAN, consulting engineer of New York, is at present conducting an investigation into the condition of the Montreal water supply conduit, before and after its collapse, for a body of prominent citizens. According to a statement he will report upon the advisability of installing a high pressure water system independent of the present waterworks plant.

J. H. HOUGH, president of the Hughes Owens Company, Limited, of Montreal, has just returned home after an extended trip to Europe and the West Indies. Mr. Hough well deserved this vacation, his first in fifteen years, during which time he has built up his business from a one-room affair to the present large company, owning valuable buildings in Toronto and Montreal, with three other important branches.

C. R. MURDOCH, B.A.Sc., has discontinued his services as waterworks engineer for Edmonton, Alta. Since August last Mr. Murdoch has been designing a filtration plant for the city, and has recently completed his work. He also had charge of the repair of the waterworks intake, which is nearing completion. Through a change of policy the position of waterworks engineer is being discontinued, the department being placed directly under the control of the City Engineer.

T. C. FORD, B.S., Ch.E., A.M., formerly chief chemist for the American Asphaltum and Rubber Company, has joined Messrs. Pullar & Enzenroth, engineering chemists of Detroit, Mich., and the firm will be known in the future as the H. B. Pullar Company. Mr. Ford has been closely associated with Mr. Pullar for the past five years, Mr. Pullar having been assistant general manager of the American Asphaltum and Rubber Company until about a year ago. Mr. Ford is not only familiar with the testing of bituminous materials, but has had wide experience in the actual building of roads and pavements. He was one of the first four engineers to complete the special post-graduate course of highway engineering at Columbia University.

REGINALD W. BROCK, Director of the Geological Survey of Canada, has been appointed Deputy Minister of Mines, to succeed DR. A. P. LOW, resigned. Professor Brock has been director of the Survey for the past twelve years, during which time his ability has been well recognized by the geological and mining authorities of the Dominion and abroad. He was previously a professor at the School of Mines, Kingston. He received his technical education at Ottawa Collegiate Institute, Queen's University, the University of Toronto, and Heidelburg. He was born in Perth forty years ago.

TORONTO BRANCH—CANADIAN SOCIETY OF CIVIL ENGINEERS.

The annual meeting of the Toronto Branch of the Canadian Society of Civil Engineers will be held on Wednesday, January 21st, 1914, at 8 o'clock p.m., in the lecture room at the Engineers' Club, 94 King Street W. Election of officers for the coming year and the presentation of annual reports will be included in the proceedings of the meeting.

COMING MEETINGS.

AMERICAN SOCIETY OF ENGINEERING CONTRACTORS.—Annual Convention to be held in New York City, January 16th, 1914. Secretary, J. R. Wemlinger, 13 Park Row, New York City.

CANADIAN SOCIETY OF CIVIL ENGINEERS.—Annual meeting will be held in Montreal, Que., January 27-29, 1914. Secretary, Prof. C. H. McLeod, 176 Mansfield Street, Montreal, Que.

AMERICAN CONCRETE INSTITUTE.—Tenth Annual Convention to be held in Chicago, February 16th to 20th, 1914. Secretary, E. E. Krauss, Harrison Building, Philadelphia, Pa.

NATIONAL CONFERENCE ON CONCRETE ROAD BUILDING.—Meeting will be held in Chicago, Ill., February 12th to 14th, 1914. Secretary, J. P. Beck, 72 W. Adams Street, Chicago, Ill.

AMERICAN SOCIETY OF HEATING AND VENTILATING ENGINEERS.—Annual Meeting to be held in New York, January 21st to 23rd, 1914. Secretary, E. A. Scott, 29 W. 39th Street, New York City.