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The Canadian Engineer

A Weekly Paper for Civil Engineers and Contractors

Reconstruction of Water Works at Cobourg, Ont.

Four Electrically-Driven Pumping Units, Each 750 G.P.M. Capacity, on Domestic Service—Gasoline-Engine-Driven Unit Replaces Steam as Standby—Three Mechanical Filter Tanks Operating Under Normal Conditions at Double Rated Capacity

By A. E. DAVISON

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COBOURG, Ont., a town of about 5,000 inhabitants, obtains its water direct from Lake Ontario. The pumping station was originally an electric power plant, to which was added a belt-driven turbine pump for supply of water to the town, with two steam pumps, of the compound duplex type, for fire service.

The system was bought by the Seymour interests when the Trent river was developed. The electrical section of the plant was then disposed of and a layout made of a motor-driven pumping plant and pressure filter system, located in the old building.

The water entered the suction well through a 12-in. intake laid upon the rock bottom of the lake and anchored thereto, and a 14-in. suction pipe with foot valve supplied the pumps from the well. In addition to this, an auxiliary supply was available from six drilled wells adjacent to the station. This water was highly mineralized, a smell of sulphur being distinctly noticeable, and on account of its corrosive quality this source of supply was only used for

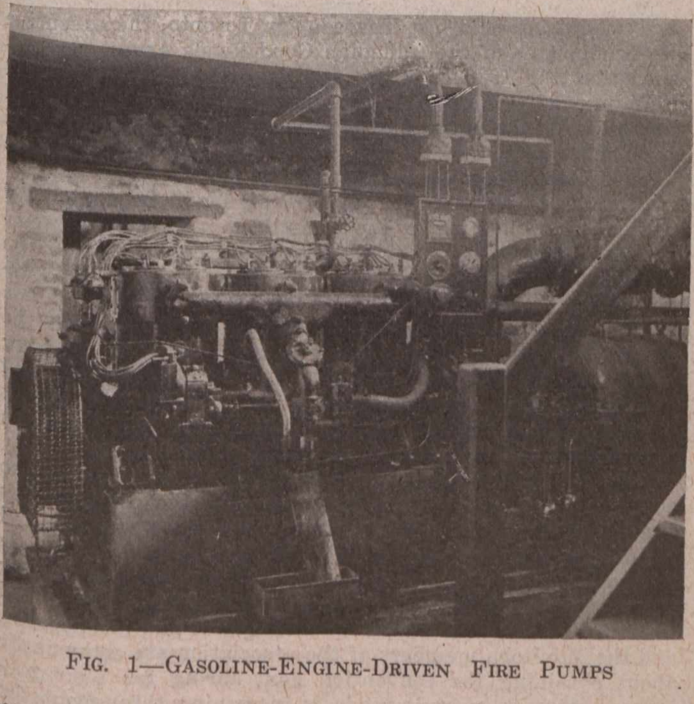

FIG. 1—GASOLINE-ENGINE-DRIVEN FIRE PUMPS

emergency and for clearing the intake of trash and ice which gave trouble in the winter season.

Four pumping units were installed, each of 750 g.p.m. capacity, arranged to operate in parallel for domestic service and in pairs in series for fire service, giving domestic capacity of 1,500 g.p.m. with reserve domestic capacity of the same amount. Pressure for domestic service is from 70 to 75 lbs., and for fire 100 lbs. at the pump-house, excess pressure under fire combination with small demand being taken care of by relief valves.

The filter plant consisted of three horizontal units 8 ft. diameter by 20 ft. long, of extra heavy construction, and with the usual connections for simple back wash. The control end of the filters is shown in Fig. 2. The rating of

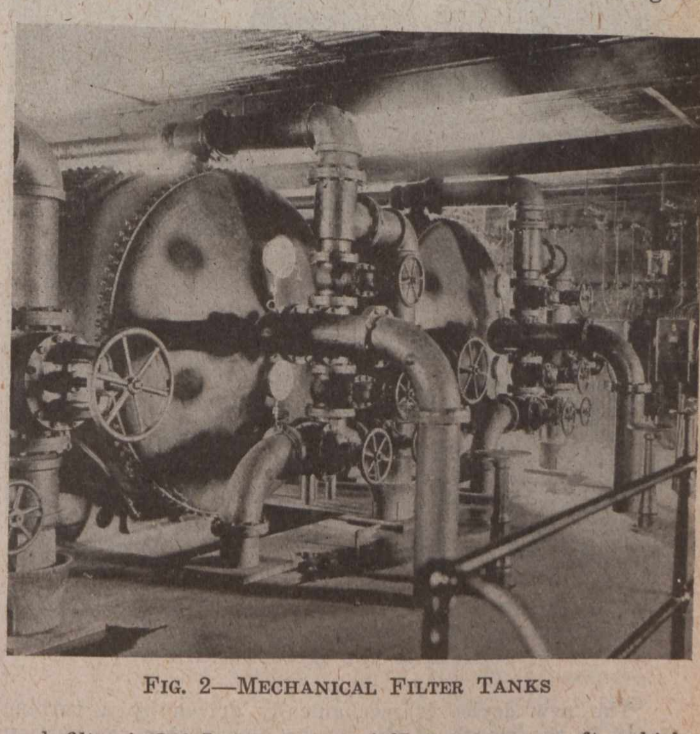

FIG. 2—MECHANICAL FILTER TANKS

each filter is 270 Imp. g.p.m. at 1.67 g.p.m. per sq. ft., which is satisfactory for muddy and contaminated water, but under present conditions, with supply taken off a clean rock bottom over 20 ft. below the surface, the quality of water is so good that the above rate can easily be increased 100%, giving a capacity of 1,600 g.p.m. under normal operation, with further increase during fire demand.

Soon after this installation was made, the town standpipe was wrecked by a fall of ice, and the system has been operated to the present time as a direct pumping proposition, without reservoir.

The location of the station is at a point on the lake shore which receives full force of all storms and there is much ice accumulation in the winter, resulting in excessive turbidity of the water near the shore and the breaking away of section of the intake pipe from time to time, so that when the system was acquired by the Hydro-Electric Power Commission of Ontario, a new intake was necessary to ensure against failure of the water supply and to relieve the filtration plant of excessive load and constant back washing at certain seasons of the year.

A new intake was completed in 1916, of riveted steel pipe, 25½ ins. diameter, 900 ft. long, laid in a channel blasted in the rock bottom of the lake and back-filled with rock, the pipe ending in a steel intake box of ample dimensions, securely anchored in about 20 ft. of water. A new suction well was blasted in the rock 10 ft. by 30 ft. by 15 ft. deep, and covered by a brick annex to the old building. The suction main was extended to this well and the old well left for the supply of the steam reserve. The cost of the intake and suction well was about $38,000. This was higher than originally estimated, due to the impossibility of maintaining drill boats, dredges and tugs at work in the lake except in unusually fine weather.

At this time an automatic device was installed for coagulant and hypochlorite in place of the usual hand-controlled feed apparatus.

Fig. 3—Automatic Device Designed by the "Hydro" Municipal Engineering Staff for Feeding Coagulant and Hypochlorite

The new device is mechanically driven by a turbine placed in the mouth of the intake pipe within the suction well, of sufficient size to operate the device under velocity head only, and with as low a discharge as 300 g.p.m.

The power required being extremely small, the speed of the turbine is proportional to the flow of water in the intake, and the device for feeding each chemical solution consists of a revolving disc on which are mounted small buckets which dip into the solution, the level of which is held constant by float control after the manner commonly obtaining in orifice boxes. The buckets discharge into a trough, from which direct pipe connection is made to the suction well. The speed of the buckets is adjustable over a wide range by friction disc drive from the turbine for any strength of solution or rate of dosing. The device arranged for two solutions is shown in Fig. 3.

Previous to the year 1917, it was necessary to hold steam at 40 lbs. pressure on the boilers as reserve for the steam pumps for fire service, and for occasions when electric power went off. The cost of this reserve becoming serious with the rise in price of coal during the war, it was decided to replace this part of the plant with two gasoline-operated units drawing water from the new suction well, and to abandon the old well and intake.

One of these units was installed about a year ago in place of one of the steam pumps, and consists of a 1,200 g.p.m. pump coupled to a six-cylinder engine, running at 1,500 r.p.m., complete with electric starting motor and storage battery, this battery being maintained in proper working condition by a small generator which can be operated at any time by belt from the coupling of one of the motor-driven domestic pumps. This unit is shown in Fig. 1.

The main gasoline tank is located in the usual manner, underground and outside the building, with the gasoline pump and auxiliary tank in a small concrete annex, an extension shaft and tell-tale passing through the wall for the purpose of operating the pump from inside the station.

The cost of this unit installed was about $7,000.

An automatic valve has been designed for cutting off the gasoline from the engine should the operator forget to do so when he shuts down the unit, in which case a leaking carburetor valve might allow gasoline to escape. This valve is controlled hydraulically from both the suction and discharge of the pump, and operates when the engine stops, independent of pressure on the pump from the town mains.

It has been sought by the above arrangement, and by fire walls and fire doors separating the electrical from the gasoline equipment, to make the gasoline reserve for fire thoroughly reliable and satisfactory when in charge of competent operators.

The total capacity for fire service under 100 lbs. pressure amounts to 3,400 g.p.m., made up of 1,200 g.p.m. for the gasoline unit, 1,700 g.p.m. for the electric units and 500 g.p.m. for the steam plant.

The steam unit is to be replaced with a second gasoline unit, and an additional unit for domestic service is under consideration.

The book value for the filter plant is $8,700, and for the electric pumping plant $9,200.

CANADIAN GOOD ROADS ASSOCIATION

At a meeting held last Thursday in Toronto, the executive committee of the Canadian Good Roads Association decided that the 1920 congress and exhibition should be held in Winnipeg. Plans were also discussed for an educational campaign for the purpose of stimulating interest in the good roads movement, and to insure that full advantage will be taken of the federal grant of $20,000,000.

Details of the educational campaign will be announced after another meeting of the committee to be held September 25th in Ottawa, but it is understood that this campaign will consist of the use of a limited amount of display advertising, and the preparation of interesting articles for use by the newspapers as reading matter. An office will be opened in Montreal and a staff engaged in order to carry on the work upon a more energetic basis. Contributions will be received from any interested firms or individuals, and it is expected that aid will also be secured from the Dominion government and the various provincial governments.

Regarding the selection of Winnipeg, it is interesting to note that this will be the first time that the congress has gone west of Toronto or Hamilton. Winnipeg was selected for the place of the convention in 1917, but owing to railway congestion and other war-time conditions, plans for holding the convention there had to be abandoned, and it was held in Hamilton instead. These difficulties have now disappeared, and the Manitoba members of the association are enthusiastic about obtaining the convention next spring for their province. Interest in good roads is increasing throughout the west. Vancouver has already requested the 1921 convention.

The executive committee of the association consists of S. L. Squire, Toronto, president; Andrew F. Macallum, Ottawa, first vice-president; Geo. A. McNamee, Montreal, secretary-treasurer; Russell T. Kelly, Hamilton, director; and J. A. Duchastel, Montreal, past president.

PUMPS FOR SMALL WATER WORKS*

By Henry A. Symonds
Boston, Mass.

IN designing a plant for pumping water for a small community, it is usually the case that the per capita cost of operating and maintenance is greater than in the large city plants, and it is important that a careful study be made of the relative merits of the different forms of power available, their adaptability, first cost, and efficiency when operating to do the particular work desired.

It is usually the case that strict economy must be practised in the introduction of these works, and studies relative to first cost have usually been made to bring the cost of the work within the ability of the communities to pay. Not so much attention, however, has been paid to the question of relative efficiency and daily cost of operation, and it frequently occurs that plants are in operation which citizens of the town, and even the water works officials themselves, believe to be highly economical, and that are the reverse. The difference in cost of operation may be so great as to make it good business to pay a much higher price for an economical unit.

Water Power—The early use of power for pumping water in the small-town installations was largely by water power or steam, and it is possible that water power is coming back into use for pumping purposes, but probably through the medium of the hydro-electric plants, as it is not common to find good water privileges so located as to be directly available for pumping from the approved sources of water-supply.

Steam Pumps—Steam has been used for many years very efficiently in many small plants.

Steam Turbines—The development of the centrifugal pump, which has now found so wide a field, was closely identified with the bringing out of the steam turbine.

The Ideal Small Pumping Plant

The centrifugal pump has been known for a great many years, but for a long time after it was invented the difficulty of getting suitable drive with sufficiently high speed retarded the development of successful operation of this type of pump. The real growth of this pump has occurred in the past ten years, during which time the use of the steam turbine and the electric motor with direct connection to the centrifugal pump has brought up the efficiency of the pump to a relatively high stage. At the present time the use of the steam turbine as applied to pumping water is mainly in the large units, and the motor is generally adopted for driving the smaller plants.

The result of the use of steam is, on the whole, satisfactory, but for the small water plants it is subject to the objections that the economical pumping machines with boilers are expensive and occupy much space, and provision for large storage of coal and adequate pumping-station buildings are required, all of which conditions add to the first cost of the plant and are aggravated by the fact that licensed engineers are required to operate such a station.

To-day we consider the ideal small pumping engine that which is economical in first cost, and is self-contained; i.e., operates from a source of power which is part of the engine or at least located in the same station. It is desirable to have it occupy as small space as is practicable, be simple in operation and not require the services of licensed men, easily repaired, quickly started, using fuel which is readily available at all times, and be capable of producing power at a relatively low unit cost.

Electric Motor—Many of the conditions of an ideal plant are met by the electric motor, and the electrical installation has become more and more common. The advantages are: Low first cost; it requires little space; low cost of maintenance; no expert supervision; it is especially well adapted to drive centrifugal pumps with direct connection, which saves friction loss of gears, and prevents noise; it may have automatic control; it may operate during periods of low load at central station, thereby getting lower rates; instant starting; as it has no reciprocating parts, it can be left to operate without attendance better than any other form of drive. The use of electricity has two serious defects in the average municipal plant; namely, it is not self-contained, but depends upon a line of wires and a power plant, usually at a distance, for its operation. For this reason neither one nor two units in a pumping station operated from the same plant are satisfactory to the insurance authorities, as they rightly claim that an accident to the wire line or the power plant will put both units out of use as quickly as one and leave the town or city in an unprotected condition in case of fire.

Attendance the Large Item of Cost

The advocates of the use of motors base their claims of low cost of operation upon the fact that attendance may be a minimum and that this is really the large item in operating small water plants. The writer believes that this argument has considerable merit, and that the improvement in efficiency brought about recently in the centrifugal pumps, and the low cost of the pump and motor, with the advantages above referred to, make this form of pumping plant a very close competitor of, and in some cases it will be found actually to be a better business proposition than, the other types of pumps and engines in spite of the difference in efficiency.

Gasoline Engine—Another form of drive which has been used in some of the small pumping plants is the gasoline engine. This form of power has practically all the advantages above mentioned except cost of operation, and in this it falls down badly, as it is operated upon a fuel of such high cost as to make the operation per horsepower-hour too high for practical purposes in the ordinary municipal water works. However, in plants where an emergency unit is wanted and electricity is not available or desirable for any reason, the gasoline engine may meet the requirements in a satisfactory manner.

Producer Gas—The gas producer as a source of power corresponds to the boiler in a steam plant. It has a great many advantages, and when combined with a proper engine for utilizing the gas to the best advantage may be considered as having most of the requirements of the ideal plant.

*From the Journal of the New England Water Works Association.

COMPARATIVE FUEL AND FIRST COSTS OF VARIOUS TYPES OF PUMPING PLANTS FOR SMALL WATER WORKS

1,500 g.p.m., 250-ft. total head; operating 300 days per year, nine hours per day; theoretical h.p., 94.7.

Pump.	Drive.	Pump efficiency per cent.	H.p. required.	Cost of fuel per h.p. hour.	Total fuel cost per hour.	Total fuel cost per 300 days.	Estimated cost of plant.	Remarks.
Vert. triplex	Fuel-oil engine	85	112	$0.004	$0.498	$1,209.60	$17,189	Guarantee—1 h.p. for 0.55 lb. fuel oil per hour. Price fuel oil, 4c. per gal.
Centrifugal	Fuel-oil engine	71	134	0.004	0.536	1,547.20	16,343	
Vert. triplex	Steam turbine	85	112	0.0075	0.840	2,268.00	13,657	Coal at $6 per ton (2,000 lbs.)
Centrifugal	Steam turbine	71	134	0.0075	1.005	2,713.50	9,957	
Vert. triplex	Electric motor	76	125	0.0124	1.55	4,185.00	11,116	Efficiency is combined pump and motor. Electricity cost, 0.9c. per h.p.-hr. plus "service charge" of 77c. per h.p., based upon max. h.p. in use, per month.
Centrifugal	Electric motor	64	148	0.0124	1.84	4,968.00	6,248	
Cross-compound, fly-wheel, steam pumping engine						1,350.00	17,000	
Compound-duplex-Deane steam pump (original plant). Rate about 700 g.p.m.						2,725.61		Actual figures of present cost. Rate of pumping about 700 g.p.m.

This form of power has been used for some time, and, so far as the writer has record, with good satisfaction in all cases. Gas produced in this way may be used in various types of internal-combustion engines with slight modification. A plant of this kind can be installed at reasonable first cost, is self-contained in so far as the power is generated directly next to the engine itself, and operates at a remarkably low cost upon a relatively inexpensive form of coal. The plant takes up more room than some of the other types, and calls for a larger pumping-station space.

Fuel-Oil Engines—In the writer's experience, in nine cases out of ten fuel-oil engines have proved an ideal installation for plants from 25 to 150 h.p., and for this reason it may be proper to give some extra details regarding this engine. The term "fuel-oil" is here used to mean any oil, from the heavier crude petroleum up to kerosene.

Fuel-oil engines are to-day known under two general heads as Diesel and Semi-Diesel, or surface ignition.

Semi-Diesel Engines

The idea conceived by Diesel is the bringing into compression a mixture of air and oil vapor to a point where heat is generated sufficient to ignite the combination. The compression reached varies from 500 to 1,100 lbs. per sq. in., but when the proper temperature occurs the gases burn rather than explode, and it is the claim of the producers of the Diesel type that this slower burning conserves the power of the gases, and the energy realized is utilized almost entirely in pushing the piston through the length of its stroke. In order to permit of the great pressure produced by this process, the machine must be exceedingly heavy, and the great amount of work which is required to perfect the Diesel engine makes it too expensive for the ordinary requirements of small water-supply plants.

The Semi-Diesel, or surface-ignition engine, on the other hand, gets its power by the explosion of a mixture of air and oil gas in the cylinder under compression around 200 to 300 lbs. per sq. in. While part of the energy is undoubtedly used in the shock against the metal of the plunger and cylinder, the resulting thrust produces motion of the piston, which is connected through the crosshead, or directly to a crank shaft which gives the motion to the driving pulley or gear.

The Semi-Diesel seems to meet all the ordinary requirements of the ideal engine described above, and while the first cost of this engine is considerably greater than that of the gasoline engine or the electric motor, the operating costs are so low that this outweighs in most cases the advantages of the low first cost of the other machines.

Poor Fuel Oils Satisfactory

In the types which are considered as small pumping outfits, designed to pump the water for communities of from 1,000 to 10,000 inhabitants, the engines required ranged from 25 to 150 h.p., but in the writer's opinion there is a large field for a still larger oil engine, and there are some machines now being produced that show wonderful efficiency in operation. They are of the Semi-Diesel type, but are able to operate on the poorest grade of fuel oil, and even tar products which have to be heated before it is possible to get them into the cylinders. The oil used runs as low as 18 degs. Beaumé, while in the smaller machines—that is, below 60 h.p.—the writer knows of good results with oil heavier than 26 degs. Beaumé, and with 25 to 35 h.p. engines kerosene or light oils of that grade have seemed to give the most satisfactory results. Pre-war prices ranged from 2 cents to 7 cents for the various grades.

The ordinary time of starting with fuel oil is from 12 to 18 min., but engines of this type may be equipped with apparatus which permits of instant starting by electric ignition and gasoline, the fuel oil being turned on after a few minutes, without interruption of the operation of the engine. This latter contrivance is of especial value where but little storage of water is possible and pumps have to be operated in case of fire, as the delay in heating the cylinder head may be serious if the supply of water is not available.

It is claimed by the advocates of the electric motor, in comparison, that the motor requires little attendance, while the oil engine calls for constant supervision. This claim is not entirely substantiated in practice, for many of the oil plants are operating for long periods of time without attendance. This of course assumes that there are duplicate units which will take care of any fire hazard if repairs are necessary. It should also be considered that constant attendance does not eliminate many of the breakdowns.

Perhaps the most satisfactory combination that can be installed for a small pumping plant for general municipal needs is made up of two duplicate units, of which it is probable that the fuel-oil engine meets the needs fully as well as any other drive which has been developed up to this time.

One advantage, of considerable importance in some cases, that steam machines have over most of the other types is in the varying of speed in operation. The speed in the electric motor and of the internal-combustion engines is variable only to a small extent, except by change gears or belt pulleys, and it is sometimes necessary to pump to waste or through a bypass back into the suction, an uneconomical process, in order to keep the rate of delivery as desired.

Comparison of Costs

The writer had occasion, a short time ago, to make a comparison of different types of pumping plants, to determine which plant would best meet the needs of a community. In connection with this, the results of investigation of different units were tabulated and are given on page 289. The figures are not to be considered as exact, either of operation or first cost, but are obtained by using quotations and guaranteed efficiencies by the manufacturers of the different lines of machinery. These figures were also taken before the extreme rise of prices which occurred since the United States entered the war, and, while they are far above the averages of five years ago, it is probable that they are nearer what we may expect in the next few years than pre-war prices.

WILLIAM ARMSTRONG RETURNS TO CANADA

WILLIAM Armstrong, who was formerly Canadian representative for a number of English manufacturers, has returned to Canada after four years of war work, and is again acting as Canadian agent for several prominent overseas firms. When war was declared, Mr. Armstrong happened to be in Vancouver. He left for New York and sailed for England, where he enlisted as a lieutenant in the admiralty, and was assigned to engineering duties on board one of the warships. Shortly afterwards, however, he was appointed technical officer in charge of mine production, and when the armistice was declared he held the rank of acting-commander. Under Mr. Armstrong's direction approximately 3,500,000 mines were manufactured, largely by electric welding.

The agencies which Mr. Armstrong has undertaken are as follows:—

Quasi-Arc Electric Welding Co., covered electrodes and resistances for use in electric welding; Sturgeon Centrifuge Co., centrifuges of various types for clay drying, sewage disposal, etc.; Hick, Hargreaves and Co., condensers, Diesel engines and steam engines; Jos. Booth Bros., Ltd., electric overhead travelling cranes; T. F. Braime and Co., seamless specialities in all metals; Yeadon, Son and Co., briquetting machinery; Head, Wrightson, Ltd., colliery apparatus and blast furnace plant; Sankey and Son, Ltd., steel wheels for automobiles; British Electric Vehicles Co., electric trucks for conveying baggage at railway terminals, handling shipments from warehouses, etc.; Jos. Carter (Stalybridge), Ltd., rubber-working machinery.

The president of the Draftsmen's Association, which has been organized in Toronto, is Geo. H. Rix, of the C.N.R. staff. The secretary is Arthur E. Fetherstonhaugh, architect, 234 Kingswood Rd., Toronto. Meetings will be held the second and fourth Fridays of each month during the fall and winter.

STREAM FLOW AND PERCOLATION WATER*

By Samuel Hall
Assistant Water Engineer, Belfast

The author, many years ago, commenced investigations of the conditions which govern and control the ever-varying flow of streams and rivers, conceiving that, even were no definite laws established, he might gain a better knowledge of the subject, and information of practical value. The author considers that the agreement between data and facts from widely different sources is such as to fully warrant the statements made in this paper. The subject is treated broadly, and in order to make it clearer many well-known conclusions are repeated.

Fig. 1 is a flow curve showing inconstant rates of discharge for a period of 24 days, and some features characteristic of probably all stream flow curves, the most noticeable being the "peaks" following rainfall, showing immediate increase of flow attributable to surface run-off. Another feature is the slowly falling parts obtaining in rainless periods, when the flow is that of percolation discharge alone. The curve shown represents the flow from a mountain catchment of between 2,000 and 3,000 acres in extent, and is taken from automatically recorded diagrams of depth of

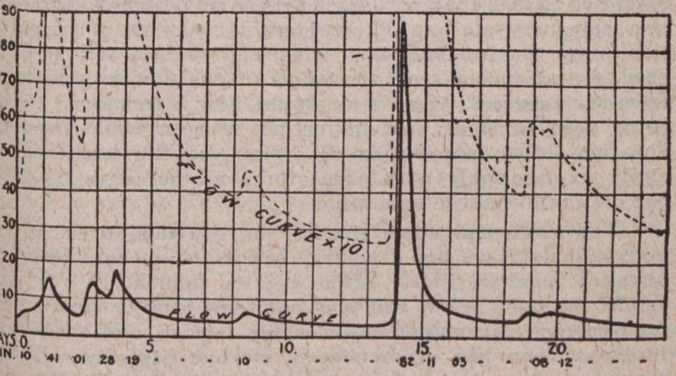

Fig. 1—Stream Flow Curve

water over a weir. It shows quick "run-off" and very marked, yet not exceptional, variations of flow.

Before comparing "peaks" of different streams, it will be interesting to compare those of one stream. They are greatest in proportion to rainfall when the surface soil is moist, always large in winter and after wet weather gradually smaller as summer advances, and smallest near the end of a long period of drought, when they contrast greatly with those of other periods. The rate of rise to a peak, when other conditions are the same, is a rough measure of rainfall intensity, which affects the height of the peak and flood rates. The falling part of the peak curve is generally slightly "slower" than the rising part, and becomes still slower as it descends. It indicates the rate of surface drainage after rain has ceased, and appears to be of some approximately regular form, but is, of course, modified by the amount of percolation discharge. The gently falling and lowest parts of the curve due to percolation discharge are generally gradually higher from the autumn and lower from spring to the end of summer, being lowest at the end of a long drought. One characteristic of these parts is the steady fall of the curve, showing not only that the rate of flow decreases, but that its rate of decrease also diminishes; in other words, the curve gets flatter from day to day throughout a rainless period. Towards the end of a drought the diminution is so small that the curve might appear to have become a horizontal line, yet examination of a greater length of curve will suffice to show such a conclusion is wrong; but it appears probable that a slowly diminishing flow might be continued during perhaps a year of further absolute drought. Some streams in these islands occasionally run dry, yet the larger streams and rivers have substantial discharges in droughts which are but of short duration compared with those which occur in some other countries. Another feature to which attention must be drawn is that the position of these parts of the curve is raised after every peak occurs. To make this clear, let us imagine a prolongation of the curve obtaining before the peak, on the line it would probably have taken had the rainfall not occurred. The produced curve would, apparently in every case, lie some distance below that which actually obtains after the peak, and the amount of difference between the produced and actual curves appears to be roughly proportionate to the extent of the peak. It seems to be clear that not only has there been a large immediate yield as shown by a peak, but that the stream has gained in "staying power." The conclusion drawn is that new supplies of percolation water have increased the amount in store, with the reslut of increased discharge. The effect of frost is, in general, precisely the same as that of absence of rain. Frost does not affect the percolation discharge, except perhaps where such occurs as small surface springs, but it holds up surface "run-off." Snow, of course, does not affect flow until thawing takes place.

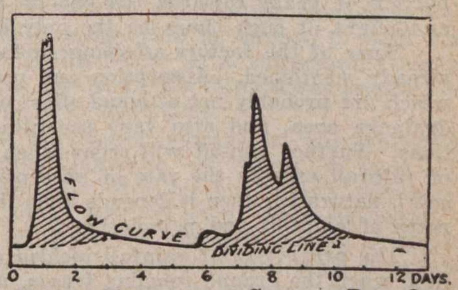

Fig. 2—Estimate of Surface Run-Off

In comparing curves of different streams it may be seen that the slope of the ground is a factor which affects the rates of rise and fall, and also the size of the "peaks," no doubt because less slope and slower "run-off" allows surface water to remain for a longer period under percolation influences. The length and slope of the stream courses are factors influencing chiefly the shape of the peaks, because after water has supplemented the flow in the upper reaches of the stream some time elapses before the effect is recorded at the gauging point. This time, roughly determinable, is a matter of hours for small streams, but of days for large rivers. Every stream course has a slight temporary storage effect, which is increased where the banks are permeable and extensive.

In order to form a rough idea of the extent to which the nature of the drainage area affects the peaks, we may assume the case of a river fed by a number of streams of similar drainage areas at various points. The effects of spates on each stream would be recorded at times, varying roughly according to the distance of the points of confluence from the gauging station. Tracing the flow curve of one stream, and altering the time for each so that if superimposed the peaks would show one after the other, the sum of these stream flow curves representing the total flow of the river, would obviously show less rates of rise and fall, and a less maximum rate in proportion to the total drainage area. Though other factors might modify the curve, the logical conclusion is that increase of size means flatter peaks and smaller maximum or flood rates in proportion to the area, whilst the converse holds for smaller areas.

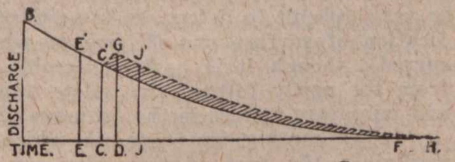

Fig. 3—Tank Discharge Curve

"Dry weather flow" is used by various engineers to mean either minimum known flow, or that for one year (average), or occasionally a supposed irreducible and constant rate which would obtain under any exceptionally adverse conditions. As a term so variously interpreted cannot lead to anything but confusion, what is meant should be expressed in other words. The minimum rate of flow is seldom the same in any different years, and any particular minimum only holds good till a lower one is recorded.

The sources of supply due to precipitation can be classed as: (1) Surface run-off; and (2) percolation discharge.

*From a paper read before the Institution of Water Engineers of Great Britain.

The chief source of deficit between rainfall on the area and resultant flow is evaporation loss. Another source of deficit is the loss of percolation water, which is inevitable, though it might be small in amount; but, on the other hand, this loss might be more than compensated for by gains of percolation water from sources outside the topographical drainage area.

(1) *Surface Run-off.*—That part of the rainfall which falls on the water surfaces as a "direct contribution" to the stream is measurable by the product of rainfall and water area. It causes the first upward turn of the curve at a peak; normally small in amount, it becomes a greater proportion of peaks towards the end of a period of drought, and might at such times be the only benefit from rainfall.

Some of the factors affecting surface run-off have been already mentioned. Absorption and percolation are factors which are probably not of equal effect over the whole of the drainage area, and also vary according to weather conditions. Surface run-off will occur when and where the rate of rainfall exceeds the rate of absorption until the surface soil is saturated; then it depends upon the difference between rates of rainfall and percolation.

The proportion of rainfall yielded as surface run-off is high when the ground remains frozen, and is generally very small in summer, sometimes very large, however, after the ground has been "baked," but whether this is actually due to the "baked" condition of the soil or to greater intensity of

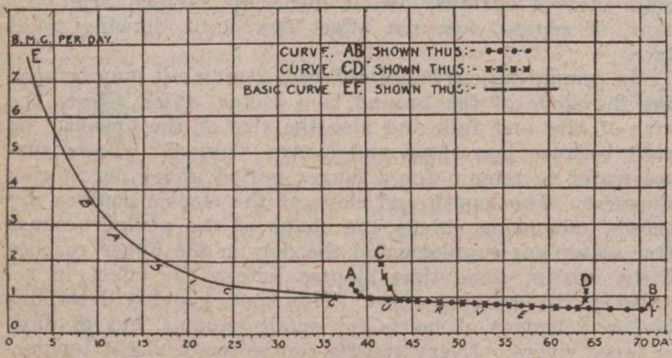

FIG. 4—BASIC CURVE OF PERCOLATION DISCHARGE

rainfall, such as obtains in thunder showers, has not been satisfactorily determined. The author has observed in ground over clay subsoil that extensive and frequent cracks develop during droughts, and it is questionable whether such cracks, at times two inches in width and more than a foot in depth, would not counter-balance the effect of baking the ground.

Whilst the ground is under the effect of recent rains, a subsequent rainfall produces a greater result in the amount of run-off. A few hours after rainfall has ceased it would be very difficult to detect, on the gathering ground, any indication of surface run-off (except where flooding has occurred), though it is quite noticeable during heavy rains from the newly formed streamlets appearing in ill-formed and miniature valleys in the drainage area.

In estimating the time at which it becomes *nil* or negligible, the fact of thin films of water being almost unnoticeable should be taken into account, and the author's estimate of the time which might elapse before the surface water has reached a stream course is given as seldom more than 48 hours, but variable according to the amount of rainfall, percolation rate, and surface slope. On this assumption dividing lines have been inserted in the flow curve in Fig. 2, to show an estimate of the increase of flow due to surface run-off; this is shown by the cross-hatched portions. By treating all the peaks in the same manner, an estimate of the yearly total of surface run-off can be made. The amount of flow, as shown by the low-lying parts of the curve, and up to the dividing line or lines is likewise an estimate of the percolation discharge. Where rainfalls succeed one another, the guide to the insertion of the dividing lines should be the estimate of percolation discharge, which has but a slowly increasing rate, and the dividing lines must obviously commence at a part of the curve which indicates percolation discharge alone. In the case of large rivers, allowance for the time taken in flowing down the river will be necessary, but percolation discharge will be the best guide for estimating purposes. It is not, of course, possible to determine accurately what proportion is due to the two intermediate sources of supply, but it will be noted from Fig. 2 that were the time of 48 hours altered to 36 or 60, it would not affect the estimate very materially.

The area from which surface run-off occurs is obviously defined by the natural water partings, but it must be noted that this area is not necessarily identical with the percolation drainage area.

Conditions Affecting Percolation Discharge

(2) *Percolation Discharge.*—Of percolation discharge there is visible and irrefutable evidence in the existence of springs discharging percolation water at the surface of the ground. There are also subaqueous springs discharging into the beds of streams, rivers and the ocean less apparent and likely to escape notice. There are doubtless similar discharges of percolation water which, from finding easy passage through alluvial deposits to open water, are perhaps outside the possibility of detection. Considering that almost all rocks are fissured, and that the beds and banks of streams are, as a general rule, extensive alluvial deposits, it is most probable that a surface discharge occurs as an exception rather than a rule. There is no better evidence of the temporary storage of considerable volumes of percolation water than stream flow when the surface soil is parched, for at such a time the whole of the flow is obviously from such sources. So far as stream flow is concerned there is no need to make any distinction between visible spring flow and other percolation discharge, as the laws which apply to one apply with equal force to the other, and to percolation discharge as a whole.

The geological conditions of the drainage area affect percolation storage and discharge, yet in the author's opinion physical conditions have often greater influence. For example, the same bed of grit may be in one locality very much fissured, and in another considerably less so, and such differences mean like differences in storage capacity. Limestone, granite, other compact rocks, and also alluvial deposits, likewise differ in storage capacity in fissures and interstices. The rate of discharge also depends probably more on physical than geological conditions, and the greater width of an outlet for percolation water might be merely accidental. The volumes of water which can be absorbed by compact rocks after they have been heated to dryness is very considerable; yet this should not be taken into account, as all the deep subsoil rocks are, and remain, moist whilst *in situ*, and the amount they can retain, despite the force of gravity, is most probably constant. For these reasons the author strongly inclines to the belief that it is the fissures, and not the compact rock, which yields the percolation discharge. There will doubtless be cases where the water percolates through compact rock; but where fissures occur, they provide much quicker drainage for the bulk of percolation water.

Percolation and Hydraulic Laws

It is not proposed to consider the subject from a geological point of view, but to consider how far percolation discharge is subject to known hydraulic laws, and in order to do that it is necessary to consider the physical conditions which affect storage and discharge. Percolation water, having passed through surface soil into permeable strata, trickles downwards through interstices and fissures until its downward progress is stopped by impermeable strata, or until it reaches an existing water surface. Its course afterwards, in either case, is most probably like that of water in open channels, to which fissures and large interstices in the strata are a rough parallel. Each fissure or miniature channel has its rate of discharge and a hydraulic gradient inseparably related to the discharge rate. The hydraulic gradient is, of course, the water surface. The surface of percolation water in a network of fissures and interstices, assumed continuous through intervening rock, has been termed the surface of saturation, but is more generally called sub-soil water level. It is really a network of hydraulic gradients, and for any particular position there is a related flow, or rate of discharge. Its slope depends partly on the size and extent of the fissures, which, being usually of small size, will have steeper gradients than those which we usually associate with open channels. The surface of saturation is not level; it bears no relation to the surface of the ground, but inclines towards the outlets for percolation water, and it coincides with free water surfaces. Its position is not fixed, but is slowly altered by discharge and supply; and should supply disturb the hydraulic gradients throughout the surface of saturation, there is a natural tendency to settlement to some position which corresponds throughout with a particular discharge rate.

The conclusions are that where part of the rainfall can percolate to the deeper subsoil, a surface of saturation is formed in the fissure and interstices of the rocks, and that there is a rate of percolation discharge corresponding to each position of the surface of saturation, and such conditions will generally obtain. Where clay immediately underlies the surface soil, discharge conditions are perhaps slightly different, but the author has not had an opportunity of investigating such a case.

Subsoil Storage Reservoirs

The conditions of storage of percolation water may not always be the same as those mentioned, for it is quite possible there are what might be termed natural subsoil, but leaking storage reservoirs, the discharge from which might be considered as from a reservoir outlet pipe. In such a case hydraulic laws would apply, and the discharge would have its related "head" and its hydraulic gradient. The author has evidence in two cases of excavations which pointed to the existence of leaking subsoil storage reservoirs. In one of these cases the volume of water was large, and the annual variation of water level small, sufficient to show that if leakage did occur it could have been continued for at least a year of absolute drought.

In considering how the available quantity of percolation water might be approximately determined from percolation discharge, it will materially assist if we first consider conditions affecting the discharge from a tank. The discharge rate or flow curve from a pipe outlet from an irregularly shaped tank would be of some such form as shown by the firm line E^1F in Fig. 3, the rate of discharge at any time being represented by an ordinate of the diagram at that time; thus E^1E represents the rate of discharge at the time E. Having the whole diagram, the total discharge is measurable; the total discharge represents the "storage capacity" of the tank, which is thus determined by means of the flow curve alone. Let us call this curve (E^1F) the basic curve, and note that any rate of discharge denotes a related position of the water level in the tank, and some corresponding amount of storage. It is quite obvious that if the tank were again filled, the basic curve would be reproduced; and if the rate of discharge at a given time be ascertained, the subsequent rates of flow, or the total available flow, can be foretold from the basic curve, no actual observations of the water levels in the tank being necessary. When a fresh supply of water is poured into the tank, it results in a corresponding alteration in the flow curve, which repeats some part of the basic curve after the supply is stopped, and by means of the basic curve the amount of supply can be determined.

Hydraulic Similarity

This case is illustrated by the dotted line in Fig. 3, which shows the effect of a fresh supply. After the supply has been stopped, the flow curve GH is a repetition of a part of the basic curve. The rate DG being the same as EE^1 at some point on the basic curve, the curve GH is the same as the curve E^1F. It is obvious that the amount of the fresh supply has caused the increase of flow shown by the shaded portion of the diagram, and the difference between the total discharges as shown by the curves C^1GH and C^1F obviously shows the total amount of the fresh supply. The latter is also measurable in another way; the ordinates EE^1 and DG are equal, and show that the quantity of water in the tank at the times E and D was the same; hence the total discharge between the times E and D as shown by the curve E^1C^1G was the net gain, or the amount of fresh supply. Similarly, by locating the time J when the ordinate JJ^1 is equal to CC^1, the total discharge between the times C and J, shown by the curve C^1GJ^1, is also equal to the fresh supply. The last mentioned is the method which appears most convenient for ascertaining the amount of gains accruing from rainfalls.

It must be borne in mind that it is a short method of measuring total gain, which, as shown by the shaded portion in Fig. 3, causes an increase of flow over a considerably longer period than that denoted by the time CJ. The aggregate percolation discharge from a drainage area is really a number of discharges from different ill-defined and overlapping areas, and may be likened to that from a series of tanks having various rates of discharge and various rates of supply, making it doubtful whether the principles would hold good; yet the average rate of supply to each tank governs the rates of discharge to a considerable extent; thus, when supplies are small low discharges prevail, and the converse of this also holds good.

The supply to percolation storage is "available rainfall," the governing influence of which is best illutrated by the fact that streams and rivers throughout the country show lowest flows at about the same times. It is now proposed to apply the above-mentioned principles, which are based upon well-known hydraulic laws, to percolation discharge, which is believed or assumed to be subject to the same laws, and should tests indicate agreement, it might reasonably be presumed that the assumption is correct.

Basic Curve of Percolation Discharge

Let us now suppose that after a period of considerable percolation the natural storage of percolation water is as great as it can be, that the stored water receives no further supply, but is exhausted by discharge. The flow curve obtained during this period, hereafter called the "basic curve of percolation discharge," would be a parallel to the basic curve of the hypothetical tank discharge; and whenever similar conditions obtain, some parts of the basic curve should be reproduced between times of fresh supplies. Let us now test the actual stream flow curve to see whether or not it does behave in this manner; for if it does, we might, from these reproduced parts, construct a considerable length of the basic curve. The method of testing is as follows: Reproduce on a diagram the flow curve shown during an absolute drought in which the lowest flow is measured, as AB in Fig. 4. (The curve in that figure is shown by a series of dots for the sake of clearness.) Reproduce a second drought flow curve in which the second lowest flow is measured, as CD, so that the ordinate showing the second lowest rate coincides with the ordinate of the first curve showing the same rate. Then, if the principles apply, a length of the second curve will coincide with a length of the first, and the coinciding portions show a length of the basic curve. Similarly, other drought or rainless period lengths of the stream flow curve can be reproduced until the greatest available length of basic curve is obtained. It is recommended that a tracing of the basic curve be then applied throughout the stream flow to test whether the flow curve in all rainless periods reproduces a part of it. The author tested in this manner the basic curve as shown by EF in Fig. 4, and found that the discrepancies were so slight that they may have been due to variations of evaporation from the stream surface. For accuracy, fine readings are necessary, particularly where the flow curve is almost flat, but in two cases the author obtained basic curves where only rough daily gaugings were given, and these showed agreement with actual flow in rainless periods as nearly as it was possible to ascertain. The stepped curve of daily gaugings was replaced by one passing through the middle points of the horizontal portions. The basic curve as shown by EF in Fig. 4 agreed with the stream flow curve from three to five days after rainfall had ceased; towards the end of this period the difference between the two was slight, but such as to shorten the times at which the basic curve was applicable; and this period was in another case apparently from four to as much as ten days, and in each case varied as the yield from rainfall was smaller or larger.

It will be obvious that the basic curve of percolation discharge is not capable of application at times when there is both surface run-off and percolation discharge, and that its use would be strictly limited to the times at which the flow curve of the stream has assumed the basic curve form, and it is important that this should be noted. When the former has assumed agreement with the latter for even a short distance, the basic curve can be used, in precisely the same manner as that obtained from the hypothetical tank discharge, to foretell subsequent rates of flow which would occur in the absence of fresh supplies to the limit of minimum flow previously measured, and to state the time at which that limit would be shown; also to determine the volume of percolation water available in the meantime. Where times of application permit, the fresh supplies form periods of rainfall, *i.e.*, the available percolation supplies, and occasionally that due to a particular rainfall, also the difference between the respective amounts of available percolation water in store at different times, can be determined. It may be noted that the length of the basic curve was determined only to certain limits, but this fact does not prevent its use. An estimate of its continuation could be made for the purpose of determining probable minimum flow under worse conditions, but otherwise would not be necessary.

When a tracing of the basic curve is applied to a flow curve at a time before a peak, to show how the latter would have continued in the absence of rain, the difference between the two curves shows the increase of flow which the rain has caused. So much of the increase as is due to increased percolation discharge would be of similar form to the shaded portion of Fig. 3. The maximum benefit from this source is shown to occur a short time after rainfall, but the benefit continues for a very long period (it might be contended, forever) at a slowly decreasing rate. As a corallary, it might be expected that the total percolation discharge at any time depends upon the amount of percolation which has obtained during many preceding months, and the effect of percolation during a particular month is reflected most largely in the next month, and to a less and less extent in each succeeding month. Examinations of many flow curves have confirmed this fact.

(*Concluded in the next issue*)

BRITISH ENGINEERING STANDARDS REPORTS

CAPT. R. J. DURLEY, secretary of the Canadian Engineering Standards Association, announces that a stock of a number of the most important publications of the British Engineering Standards Association has been received at his office, and can be obtained by any interested person upon application to him at Room 112, West Block, Ottawa. The accompanying list does not include all publications of the British Engineering Standards Association, but covers those of recent issue. Copies are for sale at twenty-five cents each excepting for the last-mentioned report (No. CL3,750) which is fifteen cents, these prices being net in all quantities. Orders for copies should be accompanied by postal note or money order payable to the Canadian Engineering Standards Association. Following is Capt. Durley's brief description of the publications now in stock:—

Report No. 15-1912. Revised August, 1912. *British Standard Specification for Structural Steel for Bridges, etc., and General Building Construction.*—This report covers process of manufacture, quality of finished steel, tensile tests, bending tests, tests on rivets, chemical analysis, inspection and other conditions.

Report No. 63-1913. *British Standard Specification for Sizes of Broken Stone and Chippings.*—This specification was formulated as a result of conferences between the quarry owners and road authorities and gives standard nomenclature, definitions and methods of measurement for broken stone and chippings.

Report No. 65-1914. *British Standard Specification for Salt-Glazed Ware Pipes.*—This report contains tables of dimensions and particulars regarding sockets, grooving, glazing, permissible variation in thickness and diameter, and methods of testing for strength and absorption.

Report No. 21-1909. Revised November, 1909. *Report on British Standard Pipe Threads for Iron or Steel Pipes and Tubes.*—This report gives definitions and tables of dimensions for British standard pipe threads. In this system the Whitworth form of thread is employed, but fine pitches are used, and both parallel and conical screw ends are provided for.

Report No. 44-1909. *British Standard Specification for Cast Iron Pipes for Hydraulic Power.*—Provision is made for two classes of this pipe, together with bends, tees and special castings. Class A: Working pressures from 700 to 900 lbs. per sq. in. Class B: Working pressures from 900 to 1,200 lbs. per sq. in. The specification covers quality of material, permissible variation of weight, marking, testing, inspection and tables of dimensions and weights.

Report No. 37-1919. Revised January, 1919. *British Standard Specification for Electricity Meters.*—This specification is intended to apply to the purchase of new meters, governing their sale by the manufacturer to the purchaser. Requirements for meters up to the largest sizes in use as well as for three-wire and three-phase meters are included. The electrolytic type of meter is not dealt with. The specification gives standard definitions and provisions regarding external characteristics, insulation, labels, standard method of marking, registering, mechanism, minimum running current, permissible limit of error and rate of registration, tests, precautions necessary in erection and other particulars.

Report No. 41-1908. *British Standard Specification for Cast Iron Spigot and Socket Flue or Smoke Pipes.*—This specification gives a schedule of dimensions and weights, with full size sections, for light cast iron spigot and socket pipes suitable for flue or smoke pipes.

Report No. 45-1917. Revised September, 1917. *Report on British Standard Dimensions for Sparking Plugs for Internal Combustion Engines.*—This report covers external dimensions only, the form of thread used being a metric thread having a 60 deg. angle. Tolerances on full diameter, effective diameter and core diameter for the thread on the plug and in the tapped hole are given, together with external dimensions of the complete plug and standard nomenclature of sparking plug parts.

Report No. 10-1904. Revised July, 1918. *British Standard Tables of Pipe Flanges.*—This report gives the British standard dimensions for pipe flanges for steam and water piping for low pressures and high pressures, dimensions of welded-on flanges for pipe lines for working steam pressures of 125, 225, and 325 lbs. per sq. in., dimensions for short flanged bends and tees of cast metal for pressures up to 325 lbs. per sq. in. and dimensions for long bends of wrought iron and steel.

Report No. 46-1909. *British Standard Specification for Keys and Keyways.*—The specification covers material, tests, definitions and tables of dimensions, for three classes of key: (a) Parallel sunk key; (b) taper key; (c) taper sunk key.

(*Concluded on page 302*)

Robert Weddell, a well-known contractor, died last week at his home in Trenton, Ont. Mr. Weddell's firm have been the contractors for a number of water-front improvements at Toronto, including the construction of the western channel and repairs to the intake.

The new million dollar laboratory of the United States Bureau of Mines will be formally opened in Pittsburgh, Pa., the end of this month. An elaborate programme of tests, visits of inspection, demonstrations of all kinds, etc., has been arranged for September 29th and 30th, and October 1st, and representatives of various universities and technical associations have been invited.

At the National Industrial Conference, which opens next Monday in Ottawa, the Association of Canadian Building and Construction Industries will be represented by: Fred Armstrong, Toronto; Col. J. A. Little, Port Arthur; H. T. Hazelton, Winnipeg; E. R. Reid, St. John, N.B.; G. H. Whitlock, Moose Jaw; and J. P. Anglin, Montreal. These gentlemen will be glad to receive any information or suggestions which will assist them in the discussion of the questions that will come before the conference.

PRESSURE RISE CAUSED BY GRADUAL GATE CLOSURE

By Norman R. Gibson
Hydraulic Engineer, Hydraulic Power Co., Niagara Falls

(Continued from last week's issue)

In the foregoing analysis no account has been taken of the effect of frictional losses of head in the penstock. As the velocity in the penstock is gradually destroyed, the friction head is gradually recovered and is added to the net head, producing discharge through the turbine gates.

Derivation of Formulas with Friction Included:—

In any penstock the total frictional losses may be assumed to be proportional to the square of the velocity of flow, and may be represented by a coefficient, F, such that FV^2 = total loss in feet of head for a velocity, V. The velocity head recovered at any time in the first interval, therefore, is $FV_0^2 - FV_{t_1}^2$. This head must then be added to the sum of $H_0 + h_{t_1}$, in determining the relation between velocity, gate opening, and head, as given by Equation (6).

Thus:—

$$V_{t_1} = (1-t_1/T)B_0(H_0+h_{t_1}+FV_0^2-FV_{t_1}^2)^{1/2}$$

Squaring:—

$$V_{t_1}^2 = (1-t_1/T)^2B_0^2(H_0+h_{t_1}+FV_0^2-FV_{t_1}^2)$$

Collecting:—

$$V_{t_1}^2[1+(1-t_1/T)^2B_0^2F] = (1-t_1/T)^2B_0^2(H_0+h_{t_1}+FV_0^2)$$

Solving and multiplying numerator and denominator by (a/g):—

$$V_{t_1} = \{ [(a/g)^2(1-t_1/T)^2B_0^2(H_0+h_{t_1}+FV_0^2)] \div [(a/g)^2+(a/g)^2(1-t_1/T)^2B_0^2F] \}^{1/2}$$

Substituting S_{t_1} for $(a/g)^2(1-t_1/T)^2B_0^2$,

$$V_{t_1} = \{ S_{t_1}(H_0+h_{t_1}+FV_0^2)/[(a/g)^2+S_{t_1}F] \}^{1/2} \quad (11)$$

TABLE 4—BY FORMULAS

Data: L = 820 ft. V_0 = 11.75 ft. per sec. H_0 = 165 ft. T = 2.1 sec. a = 4,680 ft. per sec. Friction neglected.

(1) Interval.	(2) Gate. B.	(3) Rise of pressure, h_t.	(4) Velocity, V
0	0.9148	0.0	11.7500
¼	0.8766	12.12	11.6664
½	0.8385	25.53	11.5740
¾	0.8004	40.41	11.4713
1	0.7623	56.96	11.3571
1¼	0.7242	69.94	11.1004
1½	0.6861	83.72	10.8205
1¾	0.6480	98.31	10.5149
2	0.6099	113.63	10.1805
2¼	0.5718	125.78	9.7505
2½	0.5337	138.05	9.2909
2¾	0.4956	150.32	8.8005
3	0.4574	162.42	8.2766
3¼	0.4193	171.97	7.6970
3½	0.3811	181.10	7.0899
3¾	0.3430	189.43	6.4574
4	0.3049	196.77	5.7994
4¼	0.2668	202.49	5.1146
4½	0.2287	207.14	4.4118
4¾	0.1906	211.00	3.6959
5	0.1525	213.79	2.9680
5¼	0.1143	215.75	2.2303
5½	0.0762	216.71	1.4887
5¾	0.0381	216.95	0.7446
6	0.0	216.57	0.0

Inserting this value of V_{t_1} in Equ. (5) for h_{t_1}, then

$$h_{t_1} = (a/g)V_0-(a/g)\{ S_{t_1}(H_0+h_{t_1}+FV_0^2)/[(a/g)^2+S_{t_1}F] \}^{1/2}$$

Solving for h_{t_1} in the same manner as that used in obtaining Equ. (7), and substituting R_0 in place of $(a/g)V_0$, and Z_1 in place of $(a/g)^2S_{t_1}/[(a/g)^2+S_{t_1}F]$, then:—

$$h_{t_1} = \tfrac{1}{2}\{ (2R_0+Z_1)\pm[Z_1^2+4Z_1(R_0+H_0+FV_0^2)]^{1/2} \} \quad (12)$$

In a similar manner, the equation for h_{t_2} is obtained by adding $(FV_0^2-FV_{t_2}^2)$ under the root sign in Equ. (9) and inserting the resulting value of V_{t_2} in Equ. (8). Without performing the operations, which are similar to the above, the result may be written down at once.

$$h_{t_2} = \tfrac{1}{2}\{ (2R_{t_1}+Z_2)\pm[Z_2^2+4Z_2(R_{t_1}+H_0+FV_0^2)]^{1/2} \} \quad (13)$$

Where $R_{t_1} = (a/g)V_1+h_1-C_{t_1}$

$C_{t_1} = (2a/g)(V_0-V_{t_1})$

$Z_2 = (a/g)^2S_{t_2}/[(a/g)^2+S_{t_2}F]$

and $S_{t_2} = (a/g)^2(1-t_2/T)^2B_0^2$

Similarly, the value of h, at any time in any interval, may be found.

When the gate motion is not uniform, the value S_t has to be made equal to $x^2(a/g)^2B_0^2(1-t/T)^2$, where x is a constant or variable coefficient, which may be determined by

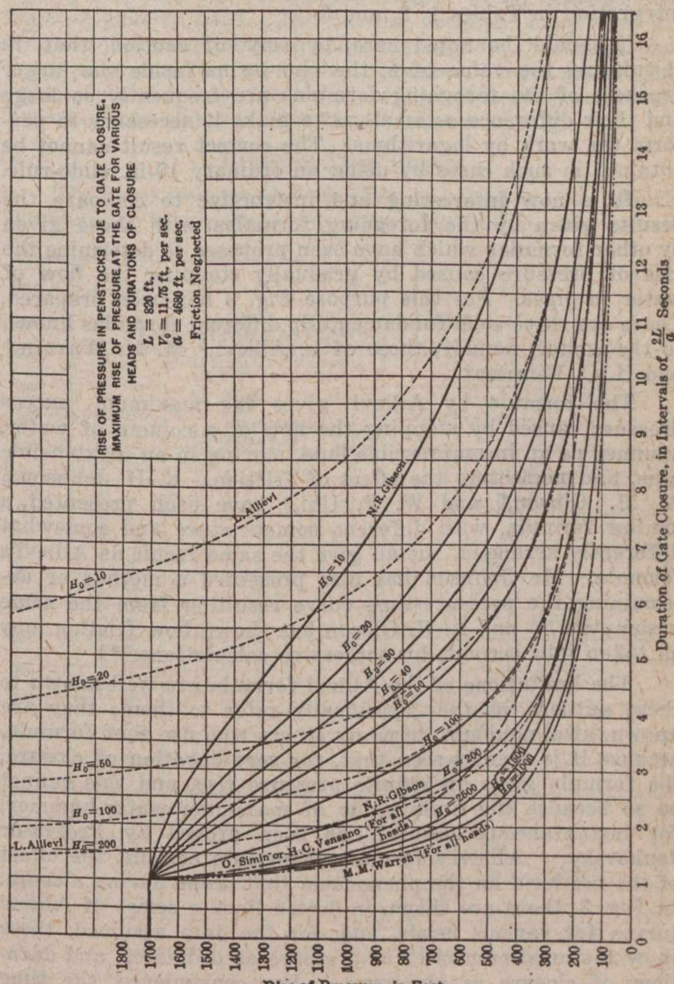

FIG. 3—MAXIMUM RISE OF PRESSURE AT THE GATE FOR VARIOUS HEADS AND DURATIONS OF CLOSURE

plotting the curve of gate opening on a time base. For uniform motion, the curve would be a straight line, the ordinate of which, at any time, t, is $(1-t/T)B_0$. For any other than uniform motion, the ordinates of the straight line would be multiplied by the constant or variable, x, which, if the motion varied in a regular manner, might sometimes be expressed in terms of t, either graphically or analytically, from the known relation between the governor movements and the

TABLE 5—BY FORMULAS

Data: L = 6,337 ft. V_0 = 15.055 ft. per sec. H_0 = 1,260 ft. T = 69.5 sec. a = 3,647 ft. per sec. Friction head, h_f = 81 ft. Non-uniform gate motion.

(1) Interval,	(2) Gate, B.	(3) Rise of pressure	(4) Velocity.
0	0.4241	0.	15.0550
1	0.4168	18.68	14.9040
2	0.4084	20.42	14.6137
3	0.3995	25.04	14.3210
4	0.3895	30.74	13.9931
5	0.3784	36.52	13.6251
6	0.3657	44.12	13.2063
7	0.3513	52.07	12.7250
8	0.3354	59.96	12.1854
9	0.3180	68.27	11.5898
10	0.2991	76.53	10.9388
11	0.2781	87.85	10.2097
12	0.2544	99.30	9.3792
13	0.2285	110.13	8.4551
14	0.2009	118.88	7.4606
15	0.1719	128.15	6.4024
16	0.1411	136.68	5.2746
17	0.1083	146.75	4.0599
18	0.0725	156.94	2.7301
19	0.0363	156.18	1.3638
20	0.0	159.21	0.

gate opening. If the motion were not regular, a graphical solution only could be obtained.

Tables 4 and 5 show the values of h_t obtained by using the formulas in the examples worked out by arithmetic integration in Tables 1, 2, and 3.

It should be noted here, by way of caution, that, in calculating the value of h, the two terms inside the larger brackets of the foregoing formulas are frequently so large and their difference so small as to make it necessary to perform the work by logarithms. The correct result cannot be obtained in such cases by using an ordinary 10-in. slide-rule.

It is now interesting and instructive to compare the results given by the foregoing formulas with those given by other formulas which have been proposed to determine the rise of pressure caused by gradually stopping the flow of water in pipes. For this purpose Fig. 3 has been prepared. There are three such fundamentally different formulas known to the writer, namely, those of L. Allievi,* M. M. Warren,† and H. C. Vensano‡.

The formula by Allievi gives the maximum excess pressure caused by stopping the flow of a column of water, assumed as an incompressible fluid, moving in an inextensible pipe, and neglecting the effect of friction. R. D. Johnson,§ A. H. Gibson,¶ and W. F. Uhl,‖ have each presented a similar formula, with different nomenclature and somewhat differently arranged, but all give the same result as Allievi's formula. Mr. Johnson has also presented a method of determining the pressure-time curve resulting from the same assumption,** and A. H. Gibson has shown how friction may be taken into account by successive calculations.***

The limitations of the Allievi formula are well known to these authors, as they specifically refer to them; they are known, also, no doubt, to many others who use that formula, because it is readily seen that, for zero duration of closure, the formula gives an infinite pressure rise, and this cannot be so because the finite value of maximum water-hammer, for instantaneous closure, has been proved by Professor Joukovsky. Allievi's formula takes into account the effect of the net head on the phenomena that occur during closure. In Fig. 3, there are shown in dotted lines a series of Allievi curves for various heads, and, for the data assumed, these show the maximum rises of pressure as ordinates, and durations of closure as abscissas. For convenience, the time abscissas have been marked off in intervals, instead of seconds. The curves, it will be noted, become very steep as the duration of closure becomes short, and, in the limit, appear

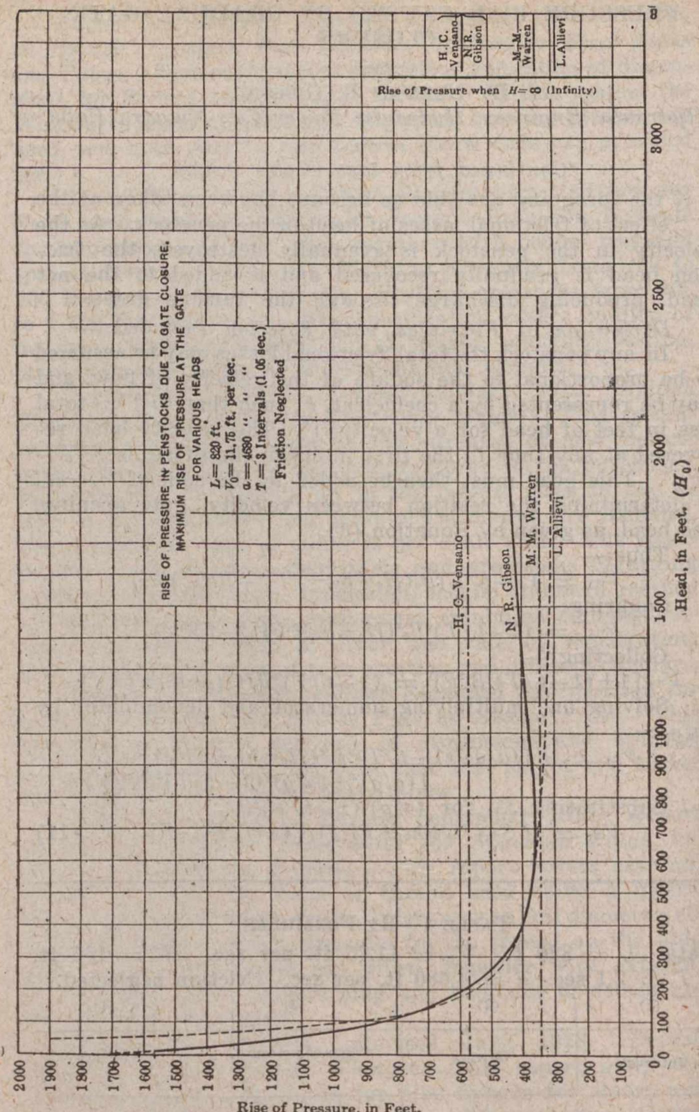

FIG. 4—MAXIMUM RISE OF PRESSURE AT THE GATE FOR VARIOUS HEADS

*The writer was unable to obtain an English translation of Mr. Allievi's paper, but his formula is commonly given as follows:—

$$h = NH/2 + H(N^2/4 + N)^{1/2}$$

where h = rise of pressure, in feet, above normal;
H = normal net head, in feet;
$N = (LV/gTH)^2$;
L = length of penstock, in feet;
V = velocity of water in penstock, in feet per second;
T = duration of gate closure, in seconds;
g = acceleration due to gravity, in feet per second per second.

†"Transactions," Am. Soc. C.E., Vol. LXXIX, pp. 238, 242. Mr. Warren's formula is $h = (LV/g)(T - L/a)$, where a = velocity of the pressure wave, and the other symbols have the same significance as in this paper.

‡"Transactions," Am. Soc. C.E., Vol. LXXIX., pp. 289-299; and Vol. LXXXII., p. 185. Mr. Vensano's formula is: $h = 2LV/gT$ with the limitation that h can never be greater than Va/g.

§"Transactions," Am. Soc. C.E., Vol. LXXIX., pp. 277-281. Mr. Johnson's formula is:

$$H_{max.} = (2MY/N^2)[M + (M^2 + N^2)^{1/2}]$$

where $H_{max.}$ $M = LV$ = maximum rise of pressure, in feet, above normal; and $N = 2gYT$, in which Y is the normal net head in feet, and the other symbols have the same significance as in this paper.

¶"Water Hammer in Hydraulic Pipe Lines," by A. H. Gibson. Mr. Gibson's formula is (in order to prevent confusion with the foregoing, some changes have been made in the nomenclature):

$$p^1 = (w/g)\{[(L/A)(A_1/T)]^2 + (L/A)(A_1/T)[2gH + (LA_1/AT)^2]^{1/2}\}$$

where p' = rise in pressure, in pounds per square foot, behind the valve at the instant where closure is complete, and therefore when p is maximum;
A = cross-sectional area of penstock, in square feet;
A_1 = maximum effective area of valve opening, in square feet;
w = weight of a cubic foot of water;
and the other symbols have the same significance as in this paper.

‖"Transactions," Am. Soc. Mech. Engrs., Paper No. 1354. Mr. Uhl's formula is:

$$DH/H = \tfrac{1}{2}n[n + (n^2 + 4)^{1/2}]$$

where DH = rise of pressure, in feet, above normal;
$n = LV/gTH$, in which the symbols have the same significance as in this paper.

**"Transactions," Am. Soc. C.E., Vol. LXXIX., pp. 277-281. Mr. Johnson's formulas, to be solved as indicated in his discussion, are:

$$t = T[(x-1)^{1/n}/x^{1/n}]$$

where $x = R\{[H + (y_0)^{1/2}]/[J - (y_0)^{1/2}]\}$,
R being $[J - (Y)^{1/2}]/[H + (Y)^{1/2}]$,
and $n = [(2gTY/LV)^2 + 1]^{1/2}$
$H = [LV/2gT(Y)^{1/2}](n-1)$
$J = [LV/2gT(Y)^{1/2}](n+1)$

***"Water Hammer in Hydraulic Pipe Lines," by A. H. Gibson.

to approach infinity as the duration of closure approaches zero. For low heads Joukovsky's limit of maximum water-hammer is passed by these curves at points of relatively long duration of gate closure. In Fig. 3, a similar series of curves for various heads has been plotted from the result obtained by using the writer's formulas, and these are shown in full lines. It will be noted that these curves show maximum water-hammer for any duration of closure from zero up to one interval $2L/a$, and, as the duration of closure becomes longer, the resulting rise of pressure finally approaches, within certain limits, the value shown by the Allievi curve. For low heads and short durations of closure, the Allievi curves generally show higher values of pressure rise than the writer's; but, for high heads and short durations of closure, they show lower values of pressure rise. This interesting fact will be referred to later when discussing the pressure-time curves.

The formulas of Messrs. Warren and Vensano do not include the net head as a factor, and, therfore, each may be represented, as shown in Fig. 3, by a single curve for all heads. Although these two formulas are objectionable, because they neglect the influence of the net head, it is evident, from the position of the curves representing them in Fig. 3, that under certain special conditions they will give, at least approximately, correct results. These conditions will be referred to later when discussing the pressure-time curves, but it is interesting to note here that Mr. Warren's formula gives approximately the correct results when the net head is such that the excess pressure will be a minimum for any

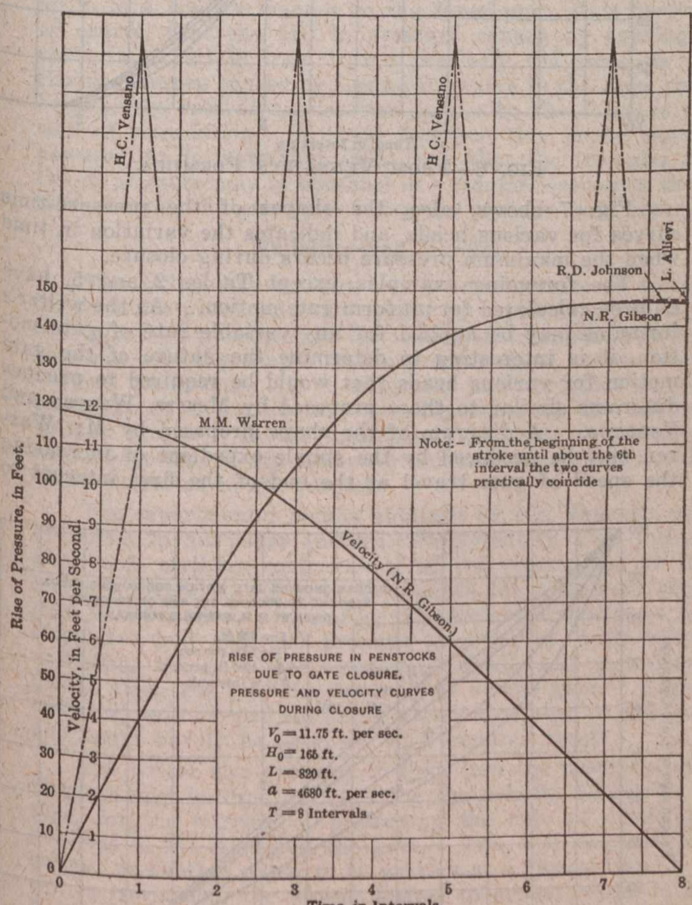

FIG. 5—PRESSURE AND VELOCITY CURVES DURING CLOSURE

given duration of closure. Mr. Vensano's formula gives approximately the correct result when the net head is very high with respect to the velocity destroyed.

Fig. 4 has been drawn to show the manner in which the rise of pressure varies with the net head. The results shown by the curves in Fig. 3 are plotted for a given duration of closure, ordinates representing excess pressure, but abscissas representing net head (H_0). It is thus seen that, for a given duration of closure, the excess pressure becomes less as the head increases, until at a certain value of H_0 the minimum rise of pressure is reached, and, as H_0 continues to increase, the rise of pressure also increases until it reaches a final value, when H_0 is infinite. In the example shown in Fig. 4, L is only 820 ft., and this length would represent the limit of gravity head. The curves, however, have been extended for higher heads, on the assumption, of course, that the head is produced by applied pressure. The corresponding curves obtained from the results of the Allievi, Warren, and Vensano formulas are also clearly shown in Fig. 4.

Pressure-Time Curves

The pressure-time curves, Figs. 5, 6, and 7, show clearly the changes in pressure that take place during the closure

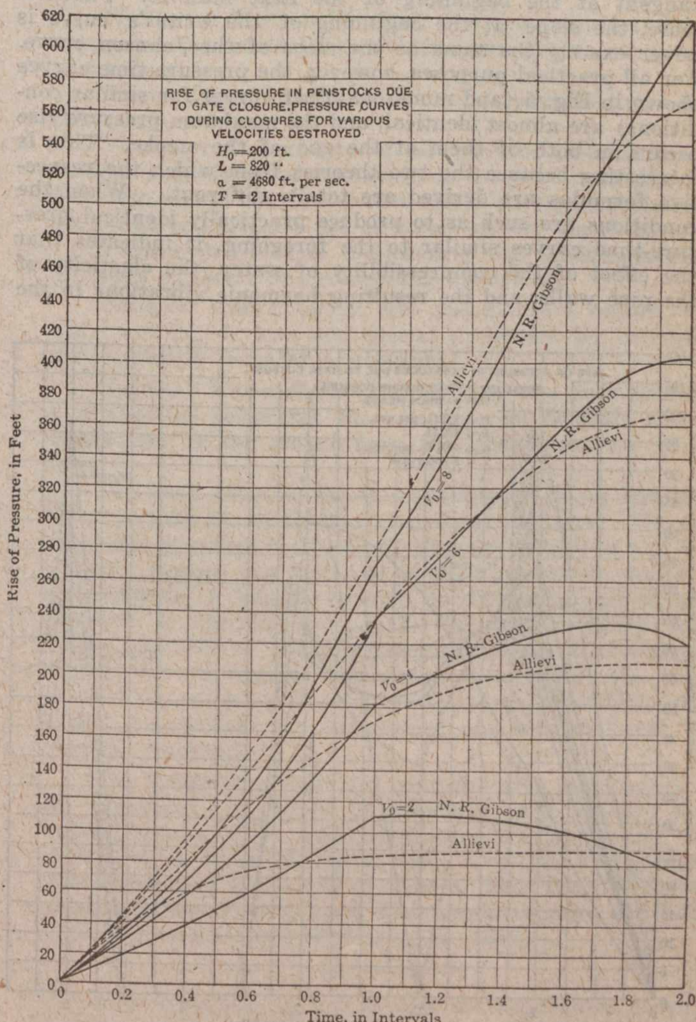

FIG. 6—PRESSURE CURVES DURING CLOSURES FOR VARIOUS VELOCITIES DESTROYED

of the gates under various typical conditions. The corresponding curves obtained from the formulas of Allievi, Warren, and Vensano have been shown in these figures for the purpose of comparison with those of the writer, and in order that the various results may be studied and explained. In some cases velocity-time curves have also been drawn, in order to show the variable rate of retardation caused by uniform gate closure.

It will be noted first that R. D. Johnson's pressure-time curve, based on Allievi's formula, or indeed Allievi's formula itself, gives the maximum pressure rise always at the end of the closing time. Mr. Warren's curve shows the excess pressure rising uniformly to a maximum at the end of the first interval and then remaining constant until the end of the closing time. Mr. Vensano's curve shows the excess pressure rising uniformly to a maximum at the end of the first interval, falling uniformly to zero at the end of the second interval, and repeating this vibration until the end of the closing time. The writer's pressure-time curves may be similar to any one of these three, depending on the duration of closure and the net head acting on the orifice, or on both of these factors.

Commencing with Fig. 5, it is seen that the pressure-time curve, under the conditions stated, is almost identical with that given by R. D. Johnson, as far as can be detected by the eye. A careful consideration of the writer's formula, however, will reveal the fact that the pressure-time curve obtained from it is not continuous, but is made up of a series of curves, each being one interval long, and cusps or changes of curvature occur at the end of each interval. Under other conditions, the cusps are plainly visible, as, for example, in Figs. 6 and 7. For any finite conditions, these cusps, theoretically, do not disappear, that is to say, the tangent at the end of one interval never exactly equals the tangent at the beginning of the next interval. Furthermore, the slope at the beginning of the writer's curve is never exactly the same as the slope of the Johnson curve. For all practical purposes, however, the pressure-time curves shown in Fig. 5 (and numerous other curves for similar conditions) are almost identical, and the maximum pressure rise occurs in both of them at the end of the stroke. This is interesting because the two theories from which the respective formulas are derived are totally different. When the conditions are such as to produce practically identical pressure-time curves similar to the foregoing, it indicates that the effect of the compressibility of water, the elasticity of the pipe walls, and the resulting harmonic vibrations in the

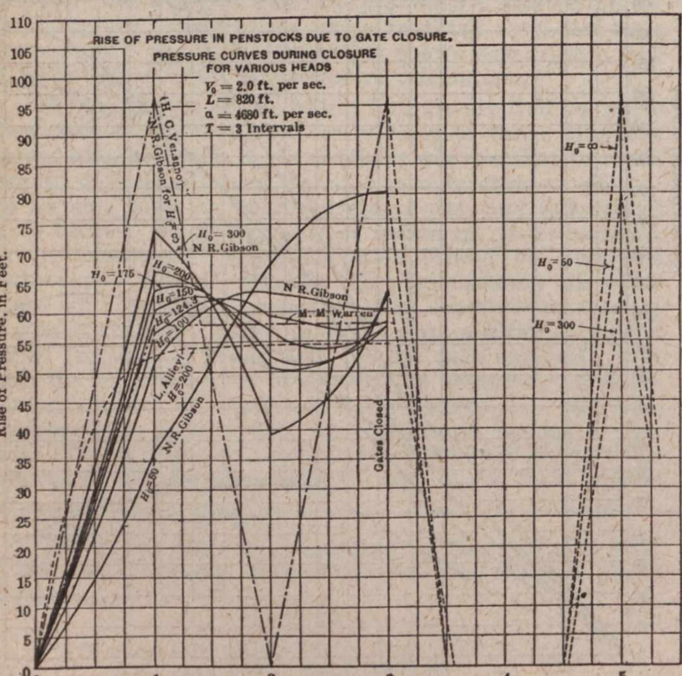

FIG. 7—PRESSURE CURVES DURING CLOSURE FOR VARIOUS HEADS

water column may be neglected, and in such cases the assumptions, on which the Allievi formula has been based, are practically sufficient.

The results obtained from the Warren and Vensano curves are much at variance with the foregoing, as indicated in Fig. 5, and the writer is of the opinion that neither of these formulas can apply for all values of H_0, and, only under certain conditions, which will be mentioned later, are they approximately correct.

As shown in Fig. 3, the maximum pressure rise given by the writer's formulas, as would be expected, is nearly always less than that given by the Allievi formula when the duration of closure is short and when H_0/V, the ratio of head to velocity destroyed, is small. When the ratio of head to velocity destroyed is large, the reverse is the case, and, when H_0 becomes infinitely large, the writer's formulas agree with that of Mr. Vensano and give results twice as great as Allievi's formula for any finite duration of closure greater than one interval. Even when the values of maximum pressure-rise, as given by the Allievi formula, are in close agreement with those of the writer, both for short and long durations of closure, the respective shapes of the pressure-time curves may be different. Figs. 6 and 7 show clearly the characteristics of the pressure-time curves in a number of typical cases, and a close study of them, together with Fig. 4, will make clear the relation that exists between the rise of pressure and the net head for any velocity destroyed. The shapes of the pressure waves that continue after the gates have been closed, until damped out by friction, are also indicated in Figs. 2 and 7. (See last week's issue for Fig. 2).

Referring now to Fig. 7, it will be noted that the particular case when Mr. Warren's formula gives approximately correct results is when the head is such that the rise of pressure due to gate closure is a minimum for the particular velocity destroyed. This occurs when $H = (3/7)(aV/g)$.

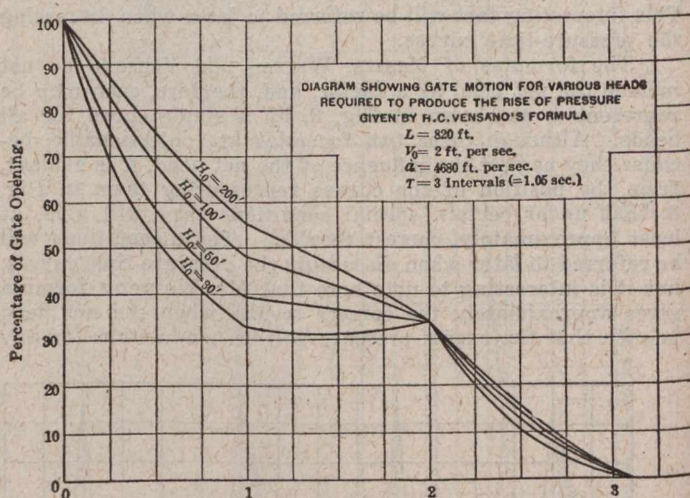

FIG. 8—FROM VENSANO'S FORMULA

Fig. 7 shows, also, the shapes of the pressure-time curves for various heads, and indicates the variation in time when the maximum pressure occurs during closure.

The foregoing examples, except Tables 2 and 5, have all been calculated for uniform gate motion. As the writer's formulas may be applied for any variable rate of gate motion, it is interesting to determine the nature of the gate motion for various heads that would be required to produce diagrams similar to those proposed by Messrs. Warren and Vensano. A diagram of the shape proposed by Mr. Warren may be produced by the simple expedient of increasing the speed of gate travel at the end of the first interval to

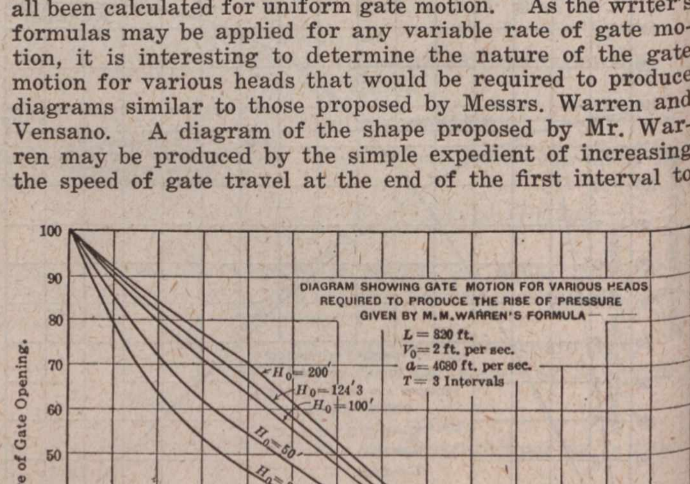

FIG. 9—FROM WARREN'S FORMULA

double the rate of its motion during the first interval and maintaining this double rate until the end of the stroke. Such an operation, though giving the same shape to the pressure-time diagram, will not, of course, give the same rise of pressure as for the same duration of uniform closure, but the latter is given by a gate motion similar to that shown in Fig. 9. If a pressure-time diagram of the same shape as

Mr. Vensano's is desired for a low head, a very complicated gate motion would be required, such, for example, as that shown in Fig. 8. In a similar manner, the gate motion required to produce any suggested form of pressure-time curve may be determined.

The manner in which excess pressures vary from gate to origin has been explained in Miss Simin's translation of Professor Joukovsky's paper, although therein worked out on the erroneous assumption of uniform retardation of the velocity of the water flowing in the pipe.

O. V. Kruse, who was associated with the writer at the time this paper was first written, has made an original study of the variation in excess pressure from gate to origin, based on the writer's application of Joukovsky's theory to slow-closing gates. The results of this study prove that when the duration of uniform closure is less than $2L/a$ the maximum pressure is exerted along the pipe to a point where the distance to the origin is equal to $Ta/2$. From that point to the origin the pressure reduces uniformly to zero. When the duration of uniform closure is equal to, or greater than, $2L/a$, the maximum rise of pressure occurs at the gate, and from there to the forebay or origin reduces to zero, uniformly along the length of the pipe.

Fall in Pressure

Professor Joukovsky's theory of pressure waves applies also to the fall in pressure produced by opening a valve or gate at the end of a pipe line, and the application of his theory to the case when a valve is gradually opened may be made in a similar manner to the foregoing. It is obvious, of course, that the fall in pressure caused by opening a valve in a certain time is not precisely the same as the rise in pressure caused by closing the valve in the same time, because the rate of acceleration cannot be the same as the rate of retardation. It would lengthen this paper unduly, however, to do more than state here that the formulas for fall in pressure may be obtained in a similar manner to those given herein, the principles being the same in both cases.

Experimental Results

The writer has not yet had an opportunity of testing the formulas presented in this paper by experiments, to show, not only the correctness of the values of maximum pressure rise, but also the shapes of the pressure-time curves under various conditions. It is expected, however, that an opportunity of doing so may possibly occur in the not too distant future. In the meantime this paper is presented in the hope that it will receive criticism or confirmation from those interested in the subject.

The experimental results obtained by Mr. Vensano, and presented in his paper entitled "Pulsations in Pipe Lines," give some striking confirmation of the correctness of the writer's formulas, not only as regards the maximum pressure rise, but also as indicating the resemblance between the calculated and observed pressure-time curves, both before and after the gate was completely closed, and at various points along the pipe line, as well as at the gate.* These, however, were carried out under a very high head (1,260 ft.), and could hardly be accepted as a general proof.

The writer also proposes to show how the pressure-time diagram may be used to determine the velocity of flow in a pipe, for the purpose of measuring the rate of discharge previous to the closing of the gate. This will be made the subject of another paper to be written at a future time.

In conclusion, the writer desires to accord credit to O. V. Kruse for the valuable help he has given in the preparation of this paper, to R. L. Hearn for the work of preparing Figs. 8 and 9, and to R. D. Johnson for the many helpful suggestions he has kindly made.

*See discussion by Norman R. Gibson in *Transactions*, Am. Soc. C. E., Vol. LXXXII., p. 236.

Jeffrey Mfg. Co., of Montreal, P.Q., and Columbus, Ohio, have opened a new branch office in Detroit, Mich., in charge of O. B. Westcott, who for many years past has been in the company's sales, engineering and construction departments.

RENSSELAER VALVE CO. APPOINTS AGENTS

ANNOUNCEMENT has been made by Irving A. Rowe, general manager of the Rensselaer Valve Co., Troy, N.Y., that he has appointed the General Supply Co. of Canada, Ltd., as sole representative for the Dominion of Canada with the exception of the city of Winnipeg and the province of British Columbia. For these two territories Mr. Rowe is under contract with other representatives.

The General Supply Co. has been rapidly building up a very strong water works department. Following are some of the firms it is now representing:—

Wallace & Tiernan Co., Inc., who are manufacturing an apparatus for controlling chlorine gas for treating water and sewage.

Electro-Bleaching Gas Co., who manufacture liquid chlorine.

Pitometer Co., makers of all types of instruments, recording and otherwise, operating on the pitot principle. This firm also makes complete water works surveys for detecting any leaks or excessive use of water on city supplies.

Pittsburgh Filter & Engineering Co., manufacturers of all types of filters and controlling apparatus.

Builders' Iron Foundry, who manufacture venturi tubes and recording devices, and special equipment for controlling and recording flows of any nature.

National Water Main Cleaning Co., who have perfected a mechanical apparatus for removing all deposits and encrustations of any nature from the interior surface of pipe lines.

The head office of the General Supply Co. is located at Ottawa, with G. B. Greene as general manager. The company has branches in Montreal, Toronto, Winnipeg and Vancouver. Among the technical staff in its water works department are J. Van Benschoten, formerly engineer with the Wallace & Tiernan Co.; R. L. Haycock, formerly water works engineer of Ottawa and superintendent of transportation with the Algoma Steel Corporation, Sault Ste. Marie; A. G. Wilkins, an engineering graduate of McGill University; and C. Laurendeau, an engineering graduate of Laval University, Montreal.

The Rensselaer Valve Co. manufactures gate valves for all pressures and types of operation and control, check valves, bronze valves, valve indicators, indicator posts, floor stands, valve boxes, air valves and Corey-type fire hydrants. The Rensselaer factory is in Cohoes, near Troy, N.Y., with sales branches in New York, Pittsburgh, Chicago and Louisville. The firm was established in 1853.

New materials for use as concrete aggregates (such as iron-ore tailings, graphite tailings and garnet tailings) are being investigated by Sub-Committee No. 7 of the Committee on Concrete and Concrete Aggregates, American Society for Testing Materials. H. S. Mattimore is chairman of the Sub-Committee.

The Canadian Engineering Standards Association has formed committees dealing with wire rope, steel bridges and their construction, rails and track, electrical work and aircraft parts, with a view to obtaining an agreement of Canadian specifications or standards that will be acceptable to nearly all concerned. In this work consideration will first be given to existing specifications or standards that have proved satisfactory in this country, Great Britain or the United States.

Lord Kelvin, lecturing to his class in Glasgow University, getting his eye on a student whose thoughts were evidently far from the class-room, pounced on this man and asked, "Will you please tell me what electricity is?" The startled student stammered and replied, "Please, sir, I have forgotten." Kelvin then turned to his class and said, "Gentlemen, behold this sad spectacle. The only man that ever knew what electricity is, has forgotten."—From an address to the London Rotary Club delivered by E. V. Buchanan, general manager of the Public Utilities Commission, London, Ont.

ROAD DRAINAGE AND FOUNDATIONS*

By Geo. Hogarth
Chief Engineer, Ontario Department of Highways

IN building any roadway, whether for light or heavy traffic, two of the most important points to be considered are drainage and foundations.

The stability of the entire road surface depends upon the foundation and the strength and life of the foundation largely depends upon the facilities provided for drainage. While proper attention may be given to the drainage of the surface of the road by giving the roadway sufficient crown to shed the water, the great importance of subsurface drainage may be entirely overlooked. There are, therefore, surface and subsurface methods of drainage and the very great importance of each must be realized and used to the fullest extent if we desire a permanent foundation. A sound and enduring foundation cannot be secured on subsoil saturated with water, but a good foundation can be secured by the simple means of keeping the subsoil dry. If water is not excluded from the foundation it will sooner or later destroy the road surfacing, since no road ever built could escape destruction should the foundation become yielding and soft.

Earth Loses Sustaining Power

The ordinary clays and earths form a naturally strong foundation when dry, but where the ground is wet and springy a good road cannot be constructed without proper subdrainage since when wet the earth loses its sustaining power and the surface formation goes to pieces. Where the subsoil is gravelly, underdrains will usually not be required, but where it is sandy, give close attention to the damp or wet spots and provide ample subdrainage.

It may be said that the whole problem of improvement and maintenance of ordinary country roads is one of drainage. Upon earth roads drainage is especially important because the material of the road surface is very susceptible to the action of water and so, more readily destroyed than are the surfaces of the better class of roads. An undrained earth road offers a splendid opportunity to the frost, and a spring thaw may result in such a road being impassable for days or weeks. A road on a wet undrained bottom will always be troublesome and expensive to maintain, and it will be economical in the long run to go to considerable expense in making the drainage of the subsoil as perfect as possible.

There are three systems of drainage, namely: Underdrainage, side ditches and surface drainage, and the objects of each will be stated.

Underdrainage will lower the water level in the soil and keep it at a safe distance from the foundations. This aids the action of the sun and wind, which tend to dry the surface of the road, but if the foundation, due to lack of underdrainage, is soft and spongy, the road will soon become a mass of mud.

Underdrainage dries the ground quickly during the spring when otherwise a saturated condition of the soil would exist.

Underdrainage intercepts the subsurface flow of water beneath the crust of the road.

Reduce Heaving and Cracking

In many cases it will be noticed that though the ground is dry, when it freezes in the fall, it is very wet and soggy in the spring when the frost comes out. The reason for this condition is that after the ground freezes, water rises slowly in the soil and if it is not drawn off by underdrainage it saturates the subsoil and rises further as the frost goes out. Underdrainage not only removes the water but reduces and will in many cases prevent the destructive heaving effects of frost.

The heaving and cracking of pavements may be entirely attributed to the freezing of water in the subgrade under the pavement. Underdrainage will greatly lessen the damage done in this way by the frost, but entirely to prevent such destructive action may be very difficult. It may be said that when underdraining a section of road known to have a wet subgrade it is well to provide a liberal number of drains. Too many cannot be laid. Particular attention to the underdrainage should be given at points on the road from 50 to 200 feet or more down grade from the top of a hill or slope and particularly so if the roadway is in excavation. Water will usually filter out to the surface at such points and the entire skill of the roadbuilder will have to be exercised in locating subdrainage to prevent heaving and cracking. In all such cases do not curtail the underdrainage, but rather put in every foot of tile that can be placed.

Wherever underdrainage is provided be careful to provide proper grade and good outlet for the tile since poor results will come from tile that are indifferently laid and improperly vented.

Tile drains furnish the usual means for subdrainage, but in some cases a blind drain may be used to good advantage. The construction of a blind drain is accomplished by opening a trench several feet deep along or across the road and afterwards filling the trench with stone. The depth, distance apart and location of such trenches will depend entirely upon the conditions encountered and the result desired. Careful attention to the subdrainage of the road if of great importance and money spent for such work is always a splendid investment.

Side ditches for all country roads are essential. Their object is to receive the water from the surface of the travelled roadway and the grade of the ditch should be such that the water is carried rapidly to a good outlet. Such ditches also intercept and carry away water that would otherwise flow from the side hills upon the road. Side ditches need not be very deep; a depth of 2½ or 3½ feet below the crown of the roadways will usually be found sufficient and the sides should be given a slope flat enough to prevent caving. In order to be effective, ditches should have as great a grade as possible, they should be well cleaned out and have free outlet in some creek or stream.

Surface drainage of the traveled portion of the road affects mainly the maintenance of the road and is accomplished by giving the road or pavement surface a crown which tends to shed the water to the side ditches. The slope of the crown should be sufficient to carry the water freely and quickly to the side ditch. The proper crown to use will vary with the nature of the roadway; when a good surface is maintained, a moderate rise will be found sufficient. For earth, clay or gravel roads a usual crown is one inch per foot, while for bituminous or concrete pavements ½ or ¼ of an inch per foot is ample.

Foundations

While drainage is undoubtedly one of the first essentials of good highways, the foundation of a road is also of great importance and one of the first substantial advances in road building was made when the laying of a course of stone for the base of a road was adopted.

The foundation of a road should normally last a number of years while the cost of maintenance should be for replacing and renewing the worn out surface. Such conditions are difficult to obtain since the foundation of 4 inches of crushed stone to-day may be entirely inadequate to-morrow under more severe traffic. The heavy loads using highways to-day require an abundance of material in the foundation of the roads and if our work is to be enduring we must see to it that strong and sound foundations are provided.

Frequently the taxpayer may enquire as to why so much material is required in a road and a demand will be made for more surface with a smaller amount of stone hidden in the foundation. Such an enquiry loses weight when the same ratepayer states that a concrete road 6 inches thick is too heavy and his idea of a concrete road is a surface of concrete about an inch thick spread over the entire space between the fences.

Roads built only a few years ago are in many cases in poor condition or destroyed to-day because a shallow, weak foundation was provided. Improved roads naturally draw traffic and people will go many miles out of their way in order to travel over a section of good road. This fact should be

*Paper presented to the Canadian Good Roads Association.

given full consideration in designing a road and any traffic census made prior to reconstruction should be considerably increased in order to estimate the travel on the improved surface. A road is not rebuilt to provide only for present traffic but it is rebuilt to carry present traffic together with whatever vehicles may use it when improved and in some cases it may be found that the traffic on an improved road immediately after reconstruction is ten or more times the original number of vehicles using the road.

Soil Determines Foundation Depth

The strength of the foundation of a road is greatly affected by the supporting power of the different soils and in order to obtain a road surface having a uniform resistance, the thickness of the crust will need to be varied. A greater depth of foundation will be required on soft clay than on gravel and for soils of intermediate strength a varying thickness will be required. Soils of poor supporting power will carry safely a load of about 600 pounds per square foot while gravel subsoil is safe under a loading of about 3,000 pounds per square foot.

Many kinds of foundation courses are available in the construction of roads, such as crushed stone, screened gravel, pit gravel, field stone, quarry stone or Telford. Before placing a foundation course the subgrade should be properly rolled in order to solidify the surface and prevent the material of the foundation being forced into the soil. This point is as important with a gravel road as it is with more permanent types of construction. A smaller amount of crushed stone or gravel will be required to build the foundation of a road on a consolidated subgrade and the rolling and compacting of the subgrade before placing the foundation is therefore an economy. A compacted subgrade also enables the foundation of the road to better withstand the effects of heavy traffic, whereas a soft and yielding subgrade tends to cause a sinkage of the foundation with consequent hollows and holes in the road.

Crushed stone where used as a foundation for a macadam road should be not less than 6 inches in thickness when compacted and the size stone will usually be about that passing a 2½ inch ring.

Screened gravel of 1 inch to 3 inch size may be used in place of crushed stone and when placed in the road a filler containing some clay or loam should be spread in order to thoroughly consolidate the stones.

Such gravel does not interlock as crushed stone does and a thicker foundation course of gravel is therefore required.

Field Stone and Telford

Field stone used for a foundation may be laid in one course from 6 inches to 12 inches in depth and the inequalities between the stones filled with coarse gravel or crushed stone. The depth of the course should in all cases be suited to the average size of field stone available as it is not economical to break them to a definite size. A variation of an inch or so in the size of the stone is of no consequence since the surface stone will serve to even up all inequalities.

The Telford base may be composed of quarry stone which has been broken to size with a stone hammer. Such foundations have given good satisfaction when used but the cost of hand placing the stones seems to have outweighed the advantages of this type of construction.

In conclusion it may be said that the adequate drainage of a road is the first requisite to permanence and if we add to proper drainage a sound foundation, the cost of maintaining the surface of the road will be greatly lessened. Drainage and foundations are two very essential details of road construction and close attention to these points when designing and constructing a road will lead to better, more durable and more satisfactory roads.

The Minister of Labor has created a Board of Conciliation to deal with the dispute between the Bedford Construction Co. of St. John, N.B., and its employees engaged on the Courtney Bay construction work. Sir Ezekiel McLeod will act as chairman, and Brig.-Gen. C. L. Hervey, of Montreal, will represent the company, and J. E. Tighe the employees.

COST-PLUS-FIXED-FEE CONTRACT

Building Construction a Huge Gamble Except Under This Form of Agreement—Co-operation for a Better Building, Plus Greater Speed and Security

By A. E. Wells

President, Wells Bros. Construction Co.

If a half dozen street urchins are caught by a burly representative of the law while engaged in the pastime of shooting craps, the law against gambling acts. Lawmakers have recognized that gambling is an unnecessary evil and have, so far as possible, put a stop to it. Yet an owner and a contractor can gamble with a million times the stake of the street urchins without fear of the law, and it is done constantly under the guise of the lump-sum contract.

Under its terms the contractor agrees that for a certain sum of money he will guarantee the owner against all the unknown conditions involved in putting up a structure. Whether he makes his figured profit or whether he loses so heavily as to be put out of business rests partly on his ability to figure costs, but largely on his luck in failing to meet those conditions which would increase costs.

Both parties to this contract stand to gain or lose. If the job costs 20% more than estimated, the owner gains to the other's loss. If conditions make considerable saving possible, then the contractor gains to the owner's loss.

Should Contractor Guarantee Cost?

Many contractors who in times past have built extensively in your field are no longer operating. Others have taken their place. Perhaps the majority of the missing firms are those who were expected not only to build according to specifications and within the time limit, but to gamble that their costs would fall within a fixed contract price. Gambling against variables such as the forces of nature and the conditions of labor—they lost. In many cases their failure involved an added investment on the part of the owner or possibly the surety company. No one gained.

The contractor is an expert retained to assemble certain materials into a finished structure. The question being asked to-day is, "Should the contractor insure the owner that his structure will not exceed a definite contract price?"

In competitive bidding the cost of this insurance is paid generally by the low bidder out of profits or, as frequently happens, out of his capital, for the reason that he is more likely to get the contract as he scales down his allowance for contingencies. In fact the man whose bid includes a safe allowance for insurance against higher costs cannot expect to obtain work under the competitive bidding system. The inevitable result is the bankruptcy of many contractors and an additional cost to the owner or the surety company to complete the unfinished contract. This situation has come to such a point that surety companies are refusing to write surety bonds on fixed-price contracts except under specially favorable conditions, and frequently recommend to owners the cost-plus-a-fixed-fee contract.

Reliable Estimate More Valuable

But from the owner's standpoint is it not preferable to know in advance what a certain project will cost? It is true that a careful estimate is due him. It should be made by a reliable contractor and checked by owner's architect and engineer. Such a figure should be more satisfactory than a competitive bid which does not necessarily show the cost of the building but only what some contractor is willing to gamble is the cost of the job.

An issue of bonds for an office building or other structure can as well be based upon a careful preliminary estimate in either case. It can only run below the estimate under the cost-plus-fixed-fee plan. Is not the owner entitled to the possible saving?

Additional financing may be an unfortunate necessity, but is there reason why the contractor should be asked to under-write the accident of greater cost?

At the Chicago meeting of the Associated General Contractors of America, this topic was discussed and Brig.-Gen. R. C. Marshall, Jr., chief of the construction division of the U.S. War Department, pointed out the fault of the usual pre-war basis of contract. He showed the impossibility on recent war work of asking for competitive bids, because speed was the essence and detailed plans and specifications were never complete at the time when construction had to start.

On such work it was, therefore, out of the question for a contractor to bid on a flat contract price basis. It would not have been fair to either side. As a result, there was developed a form of contract known as the cost-plus-sliding-scale-fee contract.

Gen. Marshall said that early in the spring of 1918, the program of work before the construction division was so extensive that it seemed advisable to have the merits of this form of contract again passed upon, and a committee of eminent business men unqualifiedly endorsed this form of contract. In Gen. Marshall's own words at the convention of general contractors:—

"Unjust, Inequitable and Uneconomic"

"No contractor should be called upon nor permitted to undertake the performance of any contract that within the four corners of the paper upon which it appears is, or may be, written the financial bankruptcy of the contractor. It is unjust, it is inequitable, it is uneconomic. The great lesson of this war on the subject of the relationship between the contractor and the owner is the cost-plus contract. This represents the only equitable basis under which a contractor may perform constructive and economic services for the owner. It is the only form of contract which affords protection to both parties. To me all the energies, the thought and the experience of this country within its own continental lines during the past year and one-half of this world struggle, shall have been in vain unless out of it shall grow, as a permanent institution, solidifying the economic relationship between the contractor and the owner, the cost-plus contract."

We have been operating under this plan almost exclusively for several years. We know that it is possible to convince most business men of the perfect fairness of the cost-plus contract, and among our clients are several who would be the last to tie themselves up with us on any basis of contract likely to be unfair or dangerous. We have built on this basis for the Robert Simpson Co., Ltd., Toronto, five separate buildings; for the William Davies Co., Ltd., Toronto, six buildings; and for Montgomery, Ward & Co., four successive times.

For these firms we have been able to start actual construction much earlier than otherwise would have been possible, which means early occupancy. The reason for this is that we can start the foundations just as quickly as the foundation plans are complete, and further design and construction may go on coincidently.

Complete Plans Not Necessary

Money tied up during construction earns nothing until the building is ready for occupancy, and the interest often amounts to a considerable sum. When we have opportunity to work with the owner, engineer and architect from the very inception of the plans, and when we begin foundations as soon as the general contour of the building and equipment are determined upon, we are able materially to cut down the period during which the owner's capital is unproductive. Under the lump-sum contract it is necessary that the plans be complete before bids are taken, which may delay occupancy for months, and without occupancy a building investment is poor as a dividend producer.

But, while speed is of first importance in most building contracts, yet fairness to both parties is an equally good reason for the general adoption of the cost-plus contract, and on that basis our company is now operating almost exclusively.

Unquestionably the contractor is called in because he is an expert in building, and not to absorb the risk entailed in the lump-sum contract. If it is not the purpose of the owner to buy price insurance along with his building, then cost-plus-fixed-fee is a better basis.

BRITISH ENGINEERING STANDARDS REPORTS

(Concluded from page 294)

Report No. 71-1917. *Report on British Standard Dimensions of Wheel Rims and Tire Bands for Solid Rubber Tires for Automobiles.*—This report gives standard sizes of wheel rims and corresponding internal dimensions of solid rubber tires, for sizes of wheel varying from 670 mm. to 881 mm. Metric dimensions are used throughout.

Report No. 72-1917. Revised September, 1917. *British Standard Rules for Electrical Machinery* (excluding motors for traction purposes).—This important report is intended to define the conditions which characterize British standard electrical machinery, including transformers but excluding traction motors, and to provide the purchaser and manufacturer with a general specification indicating the information which should be forwarded with an enquiry or an order for an electrical machine. Methods of defining the rating or rated output are formulated, and in this connection are in substantial agreement with the corresponding rules of the American Institute of Electrical Engineers. Enquiries based on these rules will enable the purchaser to compare tenders received from various manufacturers.

Report No. 74-1917. Revised September, 1917. *British Standard Specification for Charging Plug and Socket for Vehicles Propelled by Electric Secondary Batteries.*—This report contains the provisions necessary to secure interchangeability between any charging plug and any socket of the concentric type. Dimensions of the contact portion of the plug and socket, and dimensions of the gauges needed to check these are given.

Report No. 75-1916. *British Standard Specifications for Wrought Steel for Automobiles.*—This important report contains definitions of terms used, methods of testing, and specifications for ten grades of carbon, nickel and nickel-chrome steel, each specification giving chemical composition, tensile and brinell tests.

Report No. 76-1916. *British Standard Nomenclature of Tars, Pitches, Bitumens and Asphalts Used for Road Purposes and British Standard Specification for Tar and Pitch for Road Purposes.*—This valuable report defines tars, pitches, bitumens, and asphalts for road purposes, distinguishing between the tar products and bitumens and asphalts. In this respect the practice of the B.E.S.A. is not in accordance with that usual in the United States, where the term bituminous is applied in a wider sense than in Great Britain. The specification gives definitions, properties and methods of testing for two qualities of tar, and for pitch suitable for pitch-grouting.

Report No. 82-1919. *British Standard Specification for Starters for Electric Motors* (face-plate type).—This report covers definitions, pressures, methods of enclosure, standard sizes and ratings, general construction, marking and tests.

Report No. 84-1918. *Report on British Standard Fine (B.S.F.) Screw Threads and Their Tolerances.*—This report gives revised tables of dimensions for British standard fine screw threads and covers theoretical dimensions and standard sizes and tolerances of bolts and nuts for two grades of fit. The report also contains an appendix dealing with methods of determining and compensating for errors in pitch, form of thread and diameter. Much information is given regarding methods of gauging screw threads.

Report No. 88-1919. *British Standard Specification for Electric Cut-Outs for Low Pressure, Type O.*—This specification covers dimensions and standard sizes of cut-outs for low pressure and ordinary duty. A separate specification is contemplated for heavy-duty cut-outs.

Report No. CL3,750. *Interim Memorandum on French Metric Screw Threads for Aircraft Purposes.*—This memorandum describes the system of screw threads for aircraft purposes used by the French military authorities, and is accompanied by tables showing limits of size, tolerances, etc., for two grades of fit. The form of thread is that of the Systeme Internationale, in which the crest is cylindrical while the root of the thread is curved in section. The finer tolerances are provided for cases where great accuracy is required. The second grade tolerances are suitable for ordinary bolts and nuts.

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PRINCIPAL CONTENTS

ESTABLISH AN "INTERESTING CORNER"

ONE of Cleveland's leading financial institutions, the Cleveland Trust Co., whose building is at a very prominent corner in the business section of that city, has offered the use of a space adjoining its head-office building, for exhibitions of machinery and other products of Cleveland factories. The idea conceived by the Cleveland Trust Co. will prove of interest to many other municipalities where exhibitions of products manufactured locally will be of educative value and will also arouse civic pride and add to local knowledge of civic affairs. The more that citizens know of the products that are manufactured within their own town, the more likely are they to be boosters for their own municipality and to realize better its advantages for themselves and their children.

The corner which the Cleveland Trust Co. devotes to these exhibits is semi-circular, slightly above the sidewalk level, and enclosed with a low but heavy ornamental metal arilink. Each firm is allowed to continue its exhibit for a week, and there is no charge for the space. Exhibits are booked in the order in which applications for the space are made to the trust company.

The public has taken so keen an interest in these demonstrations that the spot has been popularly named "Cleveland's Most Interesting Corner." At this corner the public can obtain a close view of the operations and processes entering into the manufacture of all their home products. The exhibitions are much like those interesting ones in the Process Building at the Canadian National Exhibition, which annually proves one of the strongest attractions of that great show.

The Canadian Manufacturers' Association is spending large sums of money in advertising to the Canadian people that they should buy Canadian-made goods and so reduce our adverse trade balance. To assist this advertising campaign, one of these interesting corners should be established in every city throughout the Dominion. They would show the people, by practical demonstrations, the care and accuracy with which Canadian products are prepared, and the quality of the materials used in their manufacture. Not only would these exhibits be of benefit in increasing the sales of Canadian goods, but they would also add greatly to the knowledge possessed by Canadians of Canada's resources and manufactures.

CLASSIFICATION OF E.I.C. MEMBERS

AT present the Engineering Institute of Canada is composed of honorary members, members, associate members, juniors, students and associates. The council of the Institute might advantageously consider an amendment to the constitution and by-laws, changing the title of "member" to "fellow," and of "associate member" to "member." Should the vote favor such an amendment, the Institute would then consist of honorary members, fellows, members, juniors, students and associates.

The advantages of these changes in titles are several. One of the most important is the abolition of the misleading and inaccurate title, "associate member." The general public, and all excepting those thoroughly conversant with the Institute's affairs, are likely to think that an associate member is one who is merely associated with the Institute, and who is not an active member. As is well known within the profession, an associate member is just as much a corporate member of the Institute as is a member, but a layman is not likely to grasp the full significance of the classification, and as a result the associate members are unquestionably suffering in professional status.

We believe that the classification of "fellow" would be popular among the present "members." The word "fellow" in all scientific societies is a mark of distinction and eminence. The public are familiar with the term and are accustomed to respect it. They know that in connection with many other societies, fellowship is the highest grade of membership obtainable. There is something about the title that resembles a technical degree. A "Fellow of the Engineering Institute" would more readily obtain public respect than a "Member of the Engineering Institute." The public has always associated "fellow" with distinguished and learned men, excepting when used as a slang term. The title, "member," is so common in connection with so many different societies of a general nature for which the only entrance qualification is an annual fee, that it carries no distinction whatever. Many laymen do not realize the vast difference between being a member of a Board of Trade, for example, and being a member of the Engineering Institute of Canada. But church, political and other societies include no fellowships; the term, "fellow," is reserved by almost universal consent for learned members of respected scientific and professional societies.

There are many associate members of the Institute who could readily qualify as members but who have never applied for the transfer because the annual fees for members are heavier than for associate members. The attraction of the title, "fellow," would undoubtedly induce scores of these men to apply for transfer, and in this way the funds of the Institute could be considerably augmented in an entirely legitimate manner.

If the rather long abbreviation "A.M.E.I.C." could be abolished, there would be more uniformity in the abbreviations indicating the different degrees of membership within the Institute. F.E.I.C., M.E.I.C., J.E.I.C., S.E.I.C., and A.E.I.C. would be readily recognized as the abbreviations respectively for fellow, member, junior, student and associate.

Another advantage would be the avoidance of confusion in regard to "associate members" and "associates," another point of difference which the public does not grasp and concerning which apparently the members of the Institute themselves are often careless. Return post cards have often been mailed to members of the Engineering Institute, on which the members were supposed to write their name and the rank held in the Institute, the cards being required for branch purposes or other official work. Many associate members merely write the word "associate" after their name. On the other hand, we have knows instances where one's statement that he was an associate of the Institute was understood to mean that the speaker was an associate member, although no such meaning was intended or implied. There is a wide distinction between the requirements for membership in the two classes, but there is entirely insufficient distinction between the two titles, in all fairness to the associate members.

The classifications above suggested, including fellows and members, but containing no associate members, is the classification that was adopted some time ago by the American Institute of Electrical Engineers, and which apparently is giving excellent satisfaction among the members of that very strong organization.

The necessary amendment to the constitution and bylaws is so simple that it would not cause any confusion. It would merely be necessary to substitute the word "fellow" wherever the word "member" now appears, and the word "member" wherever the words "associate member" now appear.

PERSONALS

SIDNEY APPLETON has entered private practice in Toronto in partnership with Willard Chipman, the firm to be known as Appleton & Chipman, with offices at 58 Wellington St. East. Mr. Appleton was born January 10th, 1883, in London, Eng., and is a graduate of the East London Technical College, subsequently spending two years in an engineering course at London University. Prior to his technical education, he spent three years as an apprentice machinist, fitter and erector, and one year as a switch board operator at the Ilford Municipal Electric plant. After three years at sea as a marine engineer, Mr. Appleton was engaged for six years with R. & H. Green & Silley Weir, Ltd., as chief draftsman and manager of the Royal Albert Dock Works, London, in full charge of ship repairs. He resigned that position to become chief engineer of Trinidad Oil Fields, Ltd., in the British West Indies, where he was in charge of railway, jetty and building construction, pipe lines, power plant and all field engineering work. After one year in Trinidad, Mr. Appleton came to Canada and became town engineer of Camrose, Alta., which position he occupied for two years, resigning to become shop superintendent of the Dominion Cartridge Co., who were then engaged on war contracts, and with whom he spent two years. In June, 1917, he left the Dominion Cartridge Co. to became assistant chief engineer of the marine engine department of the Imperial Munitions Board, Ottawa, where he was in charge of the drawing office in which designs were made for use by the various shipbuilders. He requisitioned all the necessary material for the construction of the ships, and was in charge of the production of same, having under him for that purpose a large staff of executives, engineers and clerks. Mr. Appleton is an asociate member of the Institution of Mechanical Engineers of Great Britain.

J. N. FINLAYSON, professor of civil engineering, Dalhousie University, has been appointed professor of civil engineering at the University of Manitoba.

A. B. CLARSON has resigned as general secretary of the Association of Canadian Building and Construction Industries. Mr. Clarson was formerly city engineer of Verdun, P.Q. J. C. Reilly has been appointed as acting secretary, with temporary quarters at 65 Victoria St., Montreal.

W. H. TAYLOR, who has been on the staff of Frank Barber, consulting engineer, Toronto, for the past twelve years, has resigned in order to undertake water works and sewer contracting on his own account.

MAJOR N. M. HALL has been appointed lecturer in civil engineering at the University of Manitoba. Major Hall graduated in 1907 at McGill University. Until he enlisted in the Royal Engineers in 1915, he was engaged with the Western Canada Power Co. and with Ducane, Dutcher & Co., consulting engineers, Vancouver.

J. PENROSE ANGLIN, of Montreal, president of the Association of Canadian Building and Construction Industries, has just perfected the merger of the Anglin and Norcross companies, which was announced some time ago, and last week made a public issue of preferred stock in the new concern, Anglin-Norcross, Ltd. Mr. Anglin has been elected president of the merged companies.

L. P. BURNS, who recently purchased all the outstanding interests in the firm of Burns & Roberts, Ltd., Toronto, announces a change in the firm name. This company will be known in the future as L. P. Burns, Ltd., the head office address remaining the same, namely, 301 Bank of Hamilton Building, with warehouse and shops at Ritchie Ave., Toronto. The firm is manufacturing and repairing steel plate work of all kinds, and light structural work. They are also acting as agents for the Chapman Valve Mfg. Co., Pittsburgh Piping & Equipment Co., American Cement Machine Co. and the Northwest Expanded Metal Co.

The fifth general professional meeting of the Engineering Institute of Canada is being held this week in St. John, N.B.

Montreal's building permits were valued at $1,541,245 for the month of August, an increase of $482,981 as compared with August, 1918. For the eight months of this year the total was $5,913,668, a gain of $2,205,790 as compared with the same period of last year.

A. E. Doucet, director of public works of the city of Montreal, states that 33½ miles of roadways have been given one coat of bituminous oil this season, and it is his intention to apply a second coat this month. This is the first time that Montreal has used road oil instead of water as a dust preventive on macadam roads.

Frederick Law Olmstead, of Boston, Mass., will address the town planning meeting at Ottawa, October 17th and 18th, on "The Fundamental Principle of City Planning." This meeting will be jointly held by the Canadian and United States town planning institutes, and many prominent town planners and architects from the United States will likely be in attendance.

A. W. Campbell, Dominion Highways Commissioner, addressed the Union of Nova Scotia Municipalities August 27th at Yarmouth, N.S., on "Federal Assistance in Road Improvement." Among the other speakers were W. G. Clarke, chairman of the Nova Scotia Highways Board, and J. W. Roland, the board's chief engineer, both of whom discussed the work of the board in its relation to the roads of the province.