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Volume 27. Toronto, September 17, 1914

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The Canadian Engineer

A weekly paper for engineers and engineering-contractors

THE NATIONAL TRANSCONTINENTAL RAILWAY

GENERAL DESCRIPTION OF THE NATIONAL TRANSCONTINENTAL RAILWAY CONSTRUCTION, DETAILING THE METHOD OF SURVEYS EMPLOYED IN DETERMINING THE MOST DIRECT AND FEASIBLE ROUTE—STAFF ORGANIZATION, ETC.

By DUNCAN MacPHERSON,

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AN act respecting the construction of a National Transcontinental Railway was assented to by the Dominion Parliament on the 24th October, 1903, which provided for the construction of a line to be operated as a common railway highway across the Dominion of Canada, from ocean to ocean, and wholly within Canadian territory.

This line was, by the act, divided into two distinct parts; the Eastern Division, from Moncton to Winnipeg, to be constructed under a government commission, and the Western Division (extending from Winnipeg to the Pacific Ocean) to be constructed by the Grand Trunk Pacific Railway Company.

The act provides that the Eastern Division shall be built from the eastern terminus, at Moncton, through the central parts of the Province of New Brunswick, and through the Province of Quebec, by the shortest available line, to the City of Quebec; then westerly, through the northern part of the Provinces of Quebec and Ontario, and through the Province of Manitoba to the City of Winnipeg, according to such plans and specifications as may be determined, having due regard to directness, easy gradients and favorable curves.

The commissioners and the chief engineer were appointed by order-in-council August 20th, 1904, and met within a few days for organization. One of the first questions to determine was in regard to the survey work to be undertaken during the autumn and winter on that portion of territory not covered by the Grand Trunk Pacific Railway parties who were out east of Winnipeg, in the direction of, and nearly up to, Lake Abitibi. It was decided to form the territory between Moncton and near longitude 84° into four districts—"A," "B," "C," "D":—"A" from Moncton to the boundary between the Provinces of New Brunswick and Quebec, supposed to be about 290 miles; "B" from the boundary to Clear Lake, about 420 miles; "C" to the provincial boundary between Quebec and Ontario, about 300 miles; "D" to the longitude 84°, about 240 miles.

Soon after these four districts had been formed, the commissioners arranged to take over from the Grand Trunk Pacific the survey parties east of Winnipeg, with their supplies, plans, profiles, etc., and to organize two more districts, "E" and "F," thus covering the whole distance between Moncton and Winnipeg. District "E" extends about 255 miles westerly from the west boundary of "D" to a point about 30 miles west of Lake Nepigon. District "F" extends from this point to Winnipeg, about 385 miles.

The total distance from Moncton to Winnipeg was estimated to be about 1,900 miles, on what was assumed to be the most direct feasible route. The problem to be solved of definitely locating this most direct and feasible route was not an easy one, when it is remembered that, for more than half the distance, the line of general directness ran through an unsurveyed, unsettled and practically unknown region, cut up in all directions with a network of lakes and rivers, many of them not shown on any existing maps and, when so indicated, often found to be entirely misplaced. Our engineers had, therefore, in many cases, to make their own maps as the surveys proceeded and, in all cases, to correct and complete existing maps.

During the autumn of 1904 and the following spring, some 34 survey parties were equipped and sent out; and before the end of 1905 there were 45 parties in the field, consisting of about 18 men each, not counting a large number of men engaged in transporting supplies by canoe and packing in summer and by dog train in winter.

The early survey parties of 1904 were supplied on the time-honored "flour, pork and beans" food basis, and as a result of extreme cold and lack of variety in food, some of them suffered severely from scurvy.

The food schedule was revised on a liberal basis in 1905, with a plentiful assortment of soup, vegetables, jam, etc., with the result that not only was scurvy no longer heard of, but the men being so well supplied with wholesome food worked with much more energy and cheerfulness, showing that seemingly luxurious food proved a real economy.

Each survey party had an engineer in charge, transitman, leveller, topographer, draughtsman, rodman, picketman and 2 chainmen, cook and eight or nine axemen and packers. Each party was given certain governing points to connect and instructed to thoroughly exhaust the possibilities for the most favorable and reasonable direct line between these points. Barometric explorations and compass lines were followed by preliminary lines run with transit, and plans were plotted with 10-ft. contours on a scale of 400 ft. per inch. With these plans and profiles on same scale, projected locations were made on the most favorable lines and afterwards actually run on the ground and called a first location. These plans and profiles were plotted in the field, and tracings, with reports, sent to headquarters monthly. These reports were carefully gone over by the chief and assistant chief engineers, necessary changes suggested, and instructions issued accordingly. Whenever the head of a party completed what he considered the best possible first location, the engineer in charge was changed and another man given a chance to improve the line by making his best attempt at a revised location. The original head of a party, or a third man, was given a chance to still further revise for a final location. In this way it was found that a healthy rivalry was established and good results obtained. Revision of location is, however, never considered as finished until construction work is well under way; and it is often found, after the line is cleared, that slight changes will effect a very considerable saving. An equation table giving definite values for savings in distance, curvature, rise and fall, etc., was furnished all parties in the field, so that when the estimated cost of construction of any two or more lines was had, the better one to adopt from all points of view could be at once determined.

Maximum Curves.—The maximum curve used is 6° (Rad. 955 ft.) and is only used sparingly where the topographical conditions prohibit an easier radius with reasonable cost. All curves of 1° (Rad. 5,730 ft.) and sharper are connected to their tangents with easy spirals.

The maximum grades decided upon are, so far as the writer is aware, the easiest on any transcontinental line in America, being on tangents of 0.4% = 21.1 feet per mile adverse to the major or eastbound traffic, and 0.6% = 31.68 feet per mile against the comparatively minor westbound traffic. These maximum grades are used sparingly and only for the purpose of avoiding heavy work. On curves, the grades are reduced 0.04 ft. per degree in the index of the curve, so that, on the maximum curve of 6°, the maximum eastbound grade would be 0.4 minus $6 \times 0.04 = 0.16\%$ or 8.44 ft. per mile.

All curves of 1° and over are spiralled at both ends. Vertical curves are used at all intersections of grades. The width of embankments at sub-grade is 16 ft. for banks 16 ft. or less in height; the width of embankments for greater heights, 18 ft. Earth excavations are 22 ft. wide at sub-grade; rock excavations, 20 ft. wide at sub-grade. Slopes of earth embankments are 1½ to 1; rock, 1 to 1. Slopes of excavations are: earth, 1½ to 1; loose rock, 1 to 1; solid rock, ¼ to 1. The depth of ballast is 18 in. between rail base and sub-grade, or 11 in. below the under side of tie 7 in. thick.

The whole line between Moncton and Winnipeg, with the slight exception of short approaches to the Quebec Bridge on 1% grades, was definitely located with the above-mentioned very easy maximum grade; but at one point in New Brunswick, at mileage 146 from Moncton, it was found that, by the insertion of about 12½ miles of 1.1% grade adverse to eastbound traffic, a saving would be made of 17.2 miles in distance, nearly two million dollars in construction, and one and a quarter million dollars in capitalized operating value.

At another point in Quebec, near mileage 286 from Moncton, a similar grade about 10 miles long adverse to westbound traffic was found to effect a saving of 18.8 miles in distance, about half a million in construction, and over three-quarters of a million dollars in capitalized operating value. These possible temporary grades were adopted with the corresponding saving in distance and cost. If the future traffic of the road justifies the expense, these two short lengths of standard grade can be built at any time.

The proviso for directness of alignment proved a very wise precaution; as, in the Province of New Brunswick especially, the people inhabiting the fertile, well settled St. John River Valley very naturally desired to secure the advantage which would accrue to their section of country by the construction of a transcontinental railway. The fact that this would unnecessarily lengthen the line by 29 miles, a most important factor on a through route, did not appeal to them as strongly as to the inhabitants of the more western provinces anxious to secure the best possible outlets for the rapidly increasing volume of freight from the great wheat field of the west. Fortunately, our engineers were able to prove that the direct line would not only be much shorter and effect a great saving in operation, but also that the total cost of construction would be very considerably less. An additional factor in favor of the direct line was the opening up of new territory not hitherto possessing railway facilities; whereas, the St. John Valley is already served by the Canadian Pacific, and to some extent by the Intercolonial Railway.

The surveys being well advanced for some distance east of Winnipeg and west of Quebec, tenders were called, closing on the 12th March, 1906, for 150 miles of line from the north side of the St. Lawrence at Cap Rouge, westward, and for a steel viaduct 3,000 ft. long, 150 ft. high, across the Cap Rouge Valley; also for 245 miles from near Winnipeg to Peninsula Crossing, near the proposed junction with the Fort William branch of the Grand Trunk Pacific Railway. This branch line had been under construction for some time; and the intention was, as soon as it and the portion of the main line between the junction and Winnipeg were completed, to start operating between Fort William, Winnipeg and the west, thus giving another outlet to the Great Lakes from the western wheat fields.

The summer of 1906 was a busy one in railroad construction all over the continent of North America, the result being that good men were almost impossible to obtain, so that progress was not as fast as was anticipated on the two first main contracts let. The financial depression in 1907 proved, in some ways, a blessing in disguise to railway contractors; as only roads which were strong financially were able to proceed with any new construction; also, men were more plentiful. From time to time as the final location was completed on different sections, new contracts were let until, on October 29, 1908, the last contracts were let on Districts "D" and "E". In the summer of 1908, 21,000 men were at work on the various contracts between Moncton and Winnipeg.

The originally estimated distance of 1,900 miles between these points had been reduced gradually by repeated revisions of location at various points to a distance of 1,804.8 miles. This distance is 261 miles less than the shortest distance over any other combined railway between Moncton and Winnipeg. The distance between Winnipeg and Quebec City over the Transcontinental Railway, will be 1,351 miles, which is 215 miles shorter than the shortest existing line, and the grades are so much more favorable that engines of equal capacity could haul nearly twice the load on the former line than they could on the latter.

Transportation of grain by water has always been much cheaper than by rail, but the latter has been slowly and surely cheapening until at the present time, when the easy gradients and tremendously powerful locomotives of modern lines will make a combination of land difficult to excel, or peradventure, to equal on water.

The distance from Winnipeg to Quebec, via rail to Fort William, and lake, canal and St. Lawrence River to

Quebec, is 1,771 miles, involving five trans-shipments of wheat. The distance on the Transcontinental Railway will be 1,351 miles, and, as the maximum eastbound grade is 21.12 feet per mile, compensated for curvature, a heavy modern locomotive is capable of hauling on this grade a gross load behind the tender of 4,290 tons. Assuming the tare 33⅓% of gross load, the net paying load would be 2,860 tons, equal to 95,333 bushels of wheat in one train. If we assume the earnings of such trains to be $4.40 per train mile (the average earnings of the Canadian Pacific Railway freight train miles for 1913 were $2.99 for an average of 440 tons per train), we find the cost per bushel over the 1,351 miles between Winnipeg and Quebec to be 4.25 cents. The lowest rate that the writer is aware of having been in force from Fort William to Montreal, via the lake, canal and St. Lawrence River, a distance of 1,216 miles, was 4 cents per bushel in 1908. This four cents per bushel for 1,216 miles would be equivalent to 4.44 cents for 1,351 miles, so that at 4.40 per train mile, the engines above referred to could haul grain on the Transcontinental eastbound from Winnipeg to Quebec for 0.19 cents per bushel cheaper than the cheapest existing water route could haul it the same distance, and 10.86 cents per bushel cheaper than the present combined rail and water rates between the two points in question. In brief, at about one-quarter the present rail and water rate. It would appear that the days of the absolute supremacy of water transportation were in danger of at least a partial eclipse.

The comparative values of Transcontinental Railway and existing lines between Winnipeg and Quebec would be as follows:

From Winnipeg to nearest Atlantic seaport (Quebec) via Transcontinental Railway, is 1,351 miles (allowing 6 miles from north end of bridge to city) or 215 miles shorter than the shortest existing railway. Assuming the operation of only 6 daily trains each way, and the cost of a train mile $1.50, this shortening in distance alone is worth $35,312,500 in operating value capitalized at 4%, without taking into account the enormously increased earning value of the whole line due to its extremely low grades; but the maximum eastbound grades on Transcontinental being 0.4%, and westbound 0.6%, as against 1% on existing lines, not only can freight trains of about twice the capacity be hauled on former, as compared with the latter, but with the shorter distance they would make the trip in about 12 hours less time. Omitting consideration of this saving in time, which is in itself of considerable value, the operating values of the N.T.R. and existing lines can be fairly compared as follows:

Yearly cost of operating 20 daily trains on existing lines over 1,566 miles = 1,566 × 2 × 10 × 365 × $1.50	$ 17,147,700
Yearly cost on Transcontinental, 1,351 miles, is based on 12 daily trains, because they can haul the tonnage of 20 trains on existing lines = 1,351 × 2 × 6 × 365 × $1.50	8,876,070
Yearly saving in operation by N.T. Railway	$ 8,271,630
Capitalized at 4%	$206,790,750

This $206,790,750 is only the increased earning value of the Transcontinental Railway between Winnipeg and Quebec over a line with 1% grades 215 miles longer.

As the whole line from Winnipeg to Moncton is expected to cost $161,300,000, of which about $40,000,000 was expended east of Quebec and $121,300,000 west of Quebec, it would appear, therefore, that even with moderate traffic the Transcontinental is capable of earning good interest on its enormous cost.

Owing to the comparative inaccessibility of parts of the line, the last 300 miles of it were not actively under construction until the end of 1910; but by that time it was covered with workmen, rock drills, steam shovels, and all the necessaries of modern railway construction.

In order to keep check on the rate of progress of the work, the writer introduced on this line percentage forms of reports, being modifications and extensions of somewhat similar forms in use on the Canadian Pacific Railway. This form is returned monthly by the division engineers, through the district engineers; and it is then graphically plotted on a diagram which shows at a glance, not only the percentage done during the month on grading, ballasting, and all the other items of construction, but also shows the percentage done to date under each of these headings and the percentage done of the whole work in each main contract. This form of report has been found invaluable as an aid in answering requests for information from the House of Commons when in session, and for compiling our annual reports.

Our engineering organization consisted of a chief engineer, assistant chief engineer, bridge engineer, district engineers—each in charge of a district from 250 to 400 miles long—division engineers—each in charge of from 40 to 50 miles—and resident engineers—each in charge of 10 to 15 miles.

The final connection of track between Moncton and Winnipeg (except for Quebec Bridge) was made in November, 1913, and the car ferry for bridging this gap, capable of carrying 27 loaded freight cars, or a passenger train, has already arrived at Quebec.

The completion of ballasting, buildings, etc., has been pushed vigorously this season, so that it is confidently expected to have the whole line ready for transporting this season's wheat crop in October, 1914.

NEW FOREIGN TRADE COMMENCING IN AMERICA.

Gleaned from the press and from consular offices, the following items of a few of the immediate needs of China, South America and Europe should aid American manufacturers. A consular report from Rome, Italy, states that owing to all steel material for the ordnance of the ships under construction for the Italian navy being tied up in France or Germany, the navy is looking to the United States to furnish the material; and investigations have already been made among the American steel plants. One hundred and fifty thousand tons of steel are wanted. A telegram from the American minister at Caracas, Venezuela states that cement is needed in that country.

The export department of the United States Steel Corporation has received a contract for 100 miles of 80-lb. steel rails to be shipped to South America. At present Shanghai, China, is experiencing a building boom and the demand for building materials is constantly increasing, according to a consular report. At Hankow, China, there is an opportunity for the establishment of an American firm carrying a full-sized stock of building materials.

Small orders for the export of fabricated steel and machinery are already being received by American manufacturers as a direct result of the effect of the European war on industry in England and Germany. Inquiries are coming to hand at a rate that bids fair to act as a forerunner of the anticipated development of export trade, not only with South America, Africa and Australia, but with England and with Scandinavian and Mediterranean countries.

DESIGN OF NEW AND RELIEF SEWERS.

In several previous issues (August 27th and September 3rd) of *The Canadian Engineer* articles appeared dealing with special phases of the new sewerage plan for the City of Cincinnati. The sewerage investigations formed the subject of a very complete report recently issued. Another portion of it that contains an abundance of information for sewerage and municipal engineers is based on that portion of the work in connection with the design of new and relief sewers, which is chosen as the subject of this article. This work was under the direction of Mr. F. J. Van Hook, from whose report we present the following:

A critical study of the existing sewerage system of Cincinnati was made with a view to recommending such improvements as would provide relief in districts subject to flooding in times of storm, and such new sewers as may be required for districts not provided with sewerage facilities. In the design for the new sewers recommended the combined system of sewerage was employed. Since the quantity of house sewage is relatively very small, as compared with the storm water-flow, it is evident that the problem presented is one of storm sewer design.

Rainfall and Run-Off.—Before taking up the detailed studies for sewerage improvements, analyses were made of the several factors controlling the determination of the amount of surface drainage for which provision should be made in the design of storm sewers. The factors considered are as follows: (1) Intensity of rate of rainfall. (2) Relation between rainfall and run-off, or the proportion of rainfall which immediately reaches the sewers. (3) Extent and shape of the area to be drained. (4) Surface slopes.

Many attempts have been made to establish relations between these factors and to express them in formulæ. As there appeared to be no method which had been regularly and continuously used in Cincinnati for the determination of the amount of storm drainage for which sewers and drains should provide, a review was made of the more commonly accepted methods and formulæ to ascertain if any one of these was applicable to local conditions.

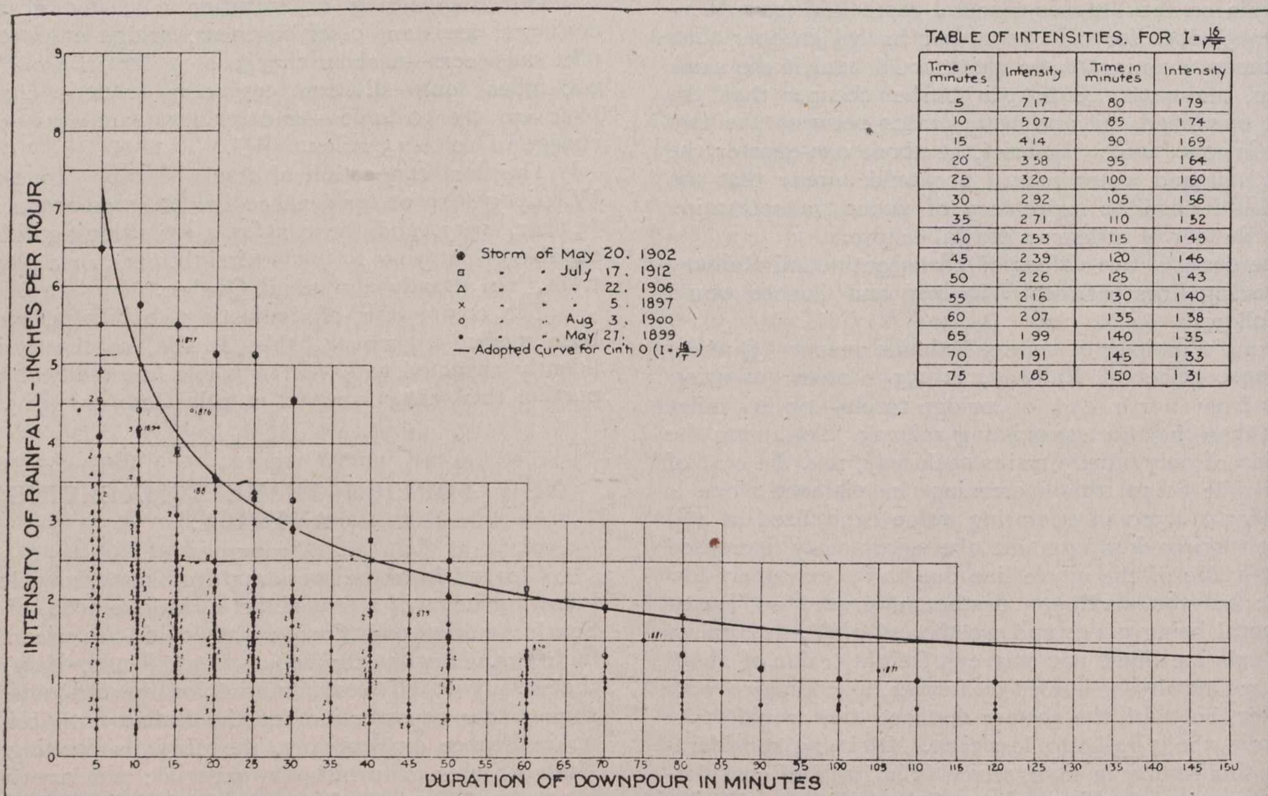
Table I.—Rates of Precipitation Which Fix Lower Limit of Excessive Precipitation in Storms Since 1904.

Period (minutes).	Rate (inches).	Period (minutes).	Rate (inches).
5	0.25	40	0.60
10	0.30	45	0.65
15	0.35	50	0.70
20	0.40	60	0.75
25	0.45	80	0.80
30	0.50	100	0.90
35	0.55	120	1.00

Most of the run-off formulæ are based on conditions existing in the particular city or locality for which they were derived and it is, therefore, evident that these should not be used excepting under similar conditions. Also, some formulæ are made dependent upon an adopted maximum rate of rainfall which is kept constant. On the other hand, the use of the so-called "rational" method of analysis through the introduction of the element of time permits of a varying rate of rainfall immediately dependent upon the area, shape and slope of the territory to be drained.

Also, on account of the unusual topographic features prevailing in Cincinnati, producing numerous small drainage areas with comparatively steep surface slopes in the outlying districts and flat slopes in the older portions of the city, it is important that the proper rainfall intensity and run-off for short periods of time should be accurately determined.

In view of these facts, it was concluded to adopt the "rational" method in determining the amount of storm



Time in minutes	Intensity	Time in minutes	Intensity
5	7 17	80	1 79
10	5 07	85	1 74
15	4 14	90	1 69
20	3 58	95	1 64
25	3 20	100	1 60
30	2 92	105	1 56
35	2 71	110	1 52
40	2 53	115	1 49
45	2 39	120	1 46
50	2 26	125	1 43
55	2 16	130	1 40
60	2 07	135	1 38
65	1 99	140	1 35
70	1 91	145	1 33
75	1 85	150	1 31

Fig. 1.—Relation Between Intensity and Duration of Rainfall, Cincinnati, 1871 to 1912.

drainage for which provision should be made. This formula may be expressed as follows:

$$Q = ACI$$

in which,

Q = the discharge in cubic feet per second.
A = the area drained in acres.
C = the ratio of run-off to rainfall.
I = the intensity or rate of rainfall during the period of concentration.

The procedure in this method of analysis is as follows:

(1) The time is first approximated by observing the surface or sewer slopes, estimating the sizes of pipe necessary and, assuming that they run full, determining the velocities. The time necessary for flow from the remote portions of the drainage area under consideration can then be calculated.

(2) With the time factor approximated, the intensity or rate of rainfall is determined from the rainfall intensity curve or equation.

(3) The run-off coefficient C, or reduction factor, is then determined by estimating the probable future increase in the percentage of impervious area.

(4) Having determined the rate of rainfall and the run-off coefficient, the discharge may be calculated and the sizes and grades of sewers determined in the usual manner. A closer estimate can then be made of the time factor and, if found necessary, a second calculation made of the sizes and grades.

Intensity or Rate of Rainfall.—The use of the "rational" method of analysis requires a thorough study of the local rainfall records for the purpose of determining a rainfall intensity curve which shall define the limit of excessive storms for which provision should be made in the design of storm sewers. By means of this curve the rate of rainfall can be determined for any period of time.

Rainfall Records.—Rainfall records for Cincinnati have been kept by the U.S. Weather Bureau from 1871 to date. Through the courtesy of the local forecaster of the weather bureau, these records have been examined and data relating to significant storms taken from them and made available for use as the basis of the rainfall studies.

The U.S. Weather Bureau published in 1912 the total monthly rainfall for each year from 1871 to 1912 inclusive, together with the total annual precipitation and the mean precipitation for each month, and for each year. These data were available but were not of particular use in the rainfall studies made in connection with the sewerage problem.

Unpublished Data Furnished by U.S. Weather Bureau.—(1) The records of heavy rainfalls having a total precipitation of 1.5 ins. or more, in 24 hours, or having a rate of 1 in. per hour for shorter periods, with extracts from the daily journal, from 1871 to 1897, were tabulated.

(2) The hourly amounts of precipitation during the storm of march 12 and 13, 1907, during which 7.19 ins. of rain fell during 48 consecutive hours were tabulated. While this was an exceptionally heavy rainfall extending over a long period of time, it did not give the very high intensities which make it of great significance in the problem of sewer design.

(3) The accumulated amounts of excessive precipitation from July 1, 1897, to July 31, 1912, for short periods of time from 5 minutes to 120 minutes were tabulated. From 1897 to 1904 the amounts were tabulated for each 5 minutes for storms in which the rate of precipitation equalled or exceeded 0.25 in. in any 5 minutes or 0.75 in. in 1 hour. After 1904 the amounts are tabulated for all storms during which the rates of precipitation equalled or exceeded in any period those given in Table I.

The data mentioned in the preceding paragraph were compiled from the charts taken from the automatic recording rain gauge at Cincinnati. These are the storms which form the most reliable basis for studies of intensity of precipitation and, while the older records furnish some valuable data the chief reliance must be placed on the records of the last 15 years.

Re-Arrangement of Data Furnished by Weather Bureau.—The table mentioned under paragraph 3, above, gives the amount of accumulated rainfall by periods ranging from 5 to 120 minutes, from the beginning to the end of excessive precipitation but does not give the maximum rainfall for each length of period indicated. A table was

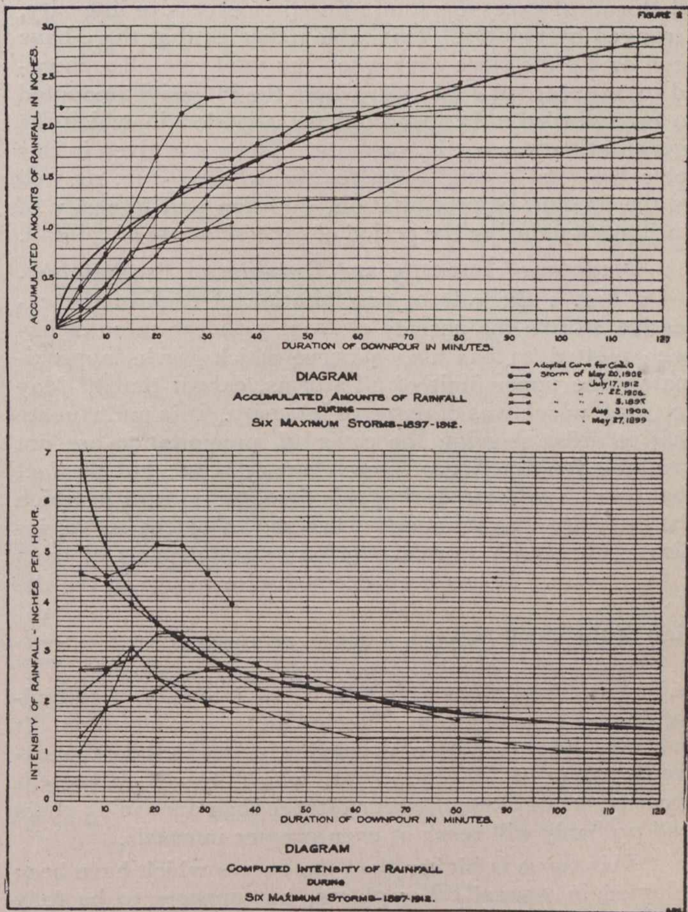


Fig. 2.

prepared to give the maximum precipitation for periods of from 5 to 120 minutes' duration. In this table the precipitation given for any specified period was the maximum amount falling during a period of such duration. For example, during the storm of September 13, 1910, the maximum quantity of precipitation during any 10 minutes was 0.41 in., whereas the amount which had fallen at the end of the first 10 minutes of excessive precipitation was but 0.21 in., and the former figure does not necessarily include the latter.

The intensity of precipitation was calculated for short periods of from 5 to 120 minutes for each of the significant storms, the result of said calculations being tabulated. These calculations are based upon the maximum quantity of precipitation during the period specified, as indicated by the data in the tabulation mentioned in paragraph 3. The intensity tabulation includes data relating to 93 storms occurring from July 5, 1897, to July 17, 1912. These data form the basis of the studies which have been made to determine the rate of rainfall for which provision should be made in designing sewers and upon which the rainfall curve hereinafter described has been based.

An examination of all records from May 31, 1871, to July 17, 1912, indicates that there have been 63 storms during which the intensity for some period of time was equal to or in excess of 1 in. per hour, and which produced a total rainfall of at least 0.5 in. during the stated period. The data relating to these storms are compiled in a table, entitled "Intensity of Precipitation During Excessive Storms at Cincinnati," and are arranged in order of descending intensities.

In another table entitled "Excessive Rainfall at Cincinnati," there have been compiled the data relating to all storms of excessive total rainfall or excessive intensity, arranged by months. This table indicates that there have been 92 storms since 1871 when the total rainfall amounted to or exceeded 2 ins., or, where the intensity amounted to or exceeded 1 in. per hour for a period in which the precipitation was 0.5 in. or more. There are given in this table the total precipitation for the whole storm, as well as the amount of precipitation, duration and intensity of precipitation during the period of excessive rate of rainfall.

Diagram of Intensity and Duration of Precipitation.—On Fig. 1 the rate of precipitation of each of the excessive storms for periods of from 5 to 150 minutes has been plotted as has also a curve which marks approximately the upper limit of all storms, except that of May 20, 1902, which was of extreme intensity. It is unfortunate that records showing the rates of precipitation are not available for a period of more than 15 years, but the fact that there have been during this time four storms in which the intensity was practically that shown by the curve, indicates that such storms may occur at intervals of from 3 to 5 years. It has been deemed wise to use in the design of the storm sewers, a curve, represented by $i = \frac{16}{T^{0.5}}$ to include these storms, in which i = intensity of precipitation and T = duration of period under consideration. It has not, however, been deemed wise to include the extreme storm of May 20, 1902, which, based on present records, is not likely to occur oftener than once in 15 years and probably will recur at even greater intervals.

This curve is higher than the curves which have been adopted in several other places, but appears to be fully warranted by the rainfall data accumulated in Cincinnati since 1871 and particularly in view of the rainfall data secured by means of the automatic recording rain gauge of the U.S. Weather Bureau during the last 15 years.

The topography of Cincinnati is such that the time of concentration in most of the sewers is very short, this making that part of the curve dealing with intensities for periods of from 5 to 30 minutes rather more important than that part dealing with intensities of periods greater than 30 minutes.

The intensities indicated by the curve have not been exceeded materially except by one storm, that of May 20, 1902. The storm of July 17, 1912, slightly exceeded the intensity indicated by the curve but the excess lies well within the limits of unavoidable errors.

In Fig. 2 the rainfall records for six maximum storms from 1897-1912 have been platted on two diagrams, the upper diagram showing the accumulated amounts and the lower diagram the actual intensity of precipitation for the successive periods of time. For purposes of comparison, the curve adopted for use in designing sewers has been shown on these diagrams.

Ratio of Run-Off to Rainfall.—Having determined upon the intensity or rate of rainfall for which provision should be made, the next factor of importance is the determination of the proportion of the rainfall which immediately reaches the sewers in any section of the city. This proportion of the ratio of run-off to rainfall is relatively difficult of determination, not only for present conditions but also for future development. Some of the factors which influence its determination are: the extent of improvement and development within the area, the surface slopes, character of the soil, frequency of storm inlets as well as the capacity of the tributary or lateral sewers to carry off the storm water.

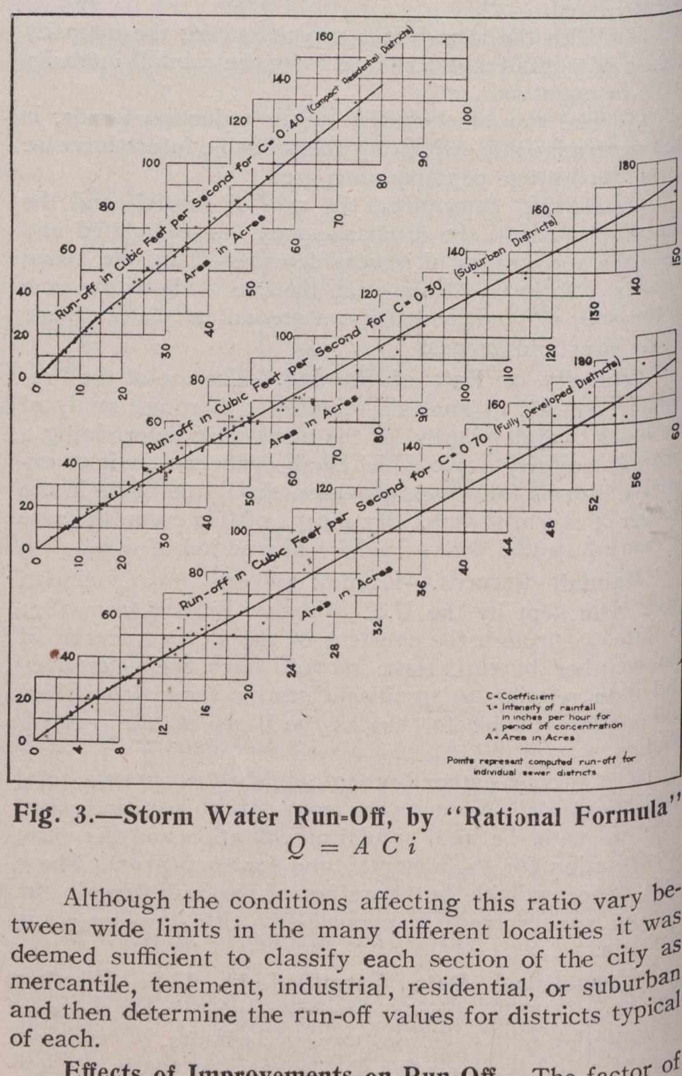


Fig. 3.—Storm Water Run-Off, by "Rational Formula"
$Q = A\,C\,i$

Although the conditions affecting this ratio vary between wide limits in the many different localities it was deemed sufficient to classify each section of the city as mercantile, tenement, industrial, residential, or suburban and then determine the run-off values for districts typical of each.

Effects of Improvements on Run-Off.—The factor of greatest importance is the extent to which each of these districts is improved and developed by buildings and pavements. To furnish the data bearing on this point surveys were made in the following districts which were selected as representing typical conditions:

(1) Mercantile district. (Walnut Street Sewer.)

(2) Tenement district. (Densely populated, Clark Street Sewer.)

(3) Combined industrial and densely populated district. (Bader Street Sewer.)

(4) Residential districts. (Walnut Hills, Bloody Run Sewer.)

In each of these districts the area covered by roofs, pavements, walks, yards, lawns, and the undeveloped territory were ascertained and to these different classes of surfaces run-off values* were applied and a final value determined upon for that district. The nature of the results obtained by these surveys and computations are shown in Tables II. to V.

Sewer Gaugings.—In addition to analyses of the surfaces of these selected typical districts, and the determination of the proportions of pervious and impervious areas, gauging stations were established in the sewers draining these districts, for the purpose of furnishing data which will serve both as a check on the method of analysis used and the relation between the rate of rainfall and the storm flow in the sewer.

To gauge the storm flows in the mercantile district an automatic clock-recording sewer gauge was placed in a side chamber of a sewer in that district. This gauge registers on a chart the depth of water flowing in the sewer during the progress of a storm. At the same time records of the rainfall are obtained by the government rain gauge located on the postoffice building, which lies within the area drained by this sewer. For the residential district a similar sewer gauge was placed in a side chamber of a sewer in that district. In order to obtain the rainfall records within this drainage area, an automatic tipping bucket rain gauge of the Frieze type was installed by the city.

The storm flow in the Clark Street Sewer, in the tenement district, is measured by means of a gauge of the maximum or flood-flow type, consisting of a staff with small bottles attached on either side which are filled as the water rises in the sewer, the highest bottle filled registering the maximum depth of flow. Other gauges of the same type were set at manholes in six other sewers.

These gauges have been maintained since June 1, 1913, but the records obtained thus far have been too meager to warrant any deductions, especially since the storms of the last five months have been of very moderate intensity and of short duration. To obtain conclusive results from storm flow gaugings it is necessary that they be extended over a long period of time in order that a sufficient number of records of storms of great intensities and sufficiently long duration may be secured. Such records will be of the greatest importance in the design of storm sewers for Cincinnati, and it is therefore to be hoped that these gaugings may be continued for several years. Fig. 4 shows details of one form of gauging chamber.

Run-Off Diagram.—Before the analyses of the improvements in typical districts were completed designs were in progress for new and relief sewers in the various parts of the city. In these designs the rational method was used and the following coefficients were adopted for the determination of the amount of run-off to be provided for:

C = .30 for suburban districts where little future development may be expected.

C = .40 for residential districts.

C = .70 for densely populated and completely developed tenement and small block districts.

The amounts of run-off which were determined in the course of the numerous designs have been plotted and curves projected by the use of which run-off quantities can be determined directly for any area in question without otherwise laborious computation work. These curves, shown on Fig. 3, have been used for the later preliminary studies. For completely developed commercial districts higher coefficients should be used.

Method of Study.—With the completion of these preliminary studies, and decision as to method of design, the preparation of plans for new and relief sewers was at once started, the problems generally recognized as most urgent being attacked first.

Data Available.—The complaints received were used as an index to the localities or districts in most urgent need of relief sewerage. For the study of these districts it was necessary to have at hand the records of existing sewers, as well as the proper portions of the topographic map. In so far as possible the two branches of the Division of Sewerage Investigations which were carrying on

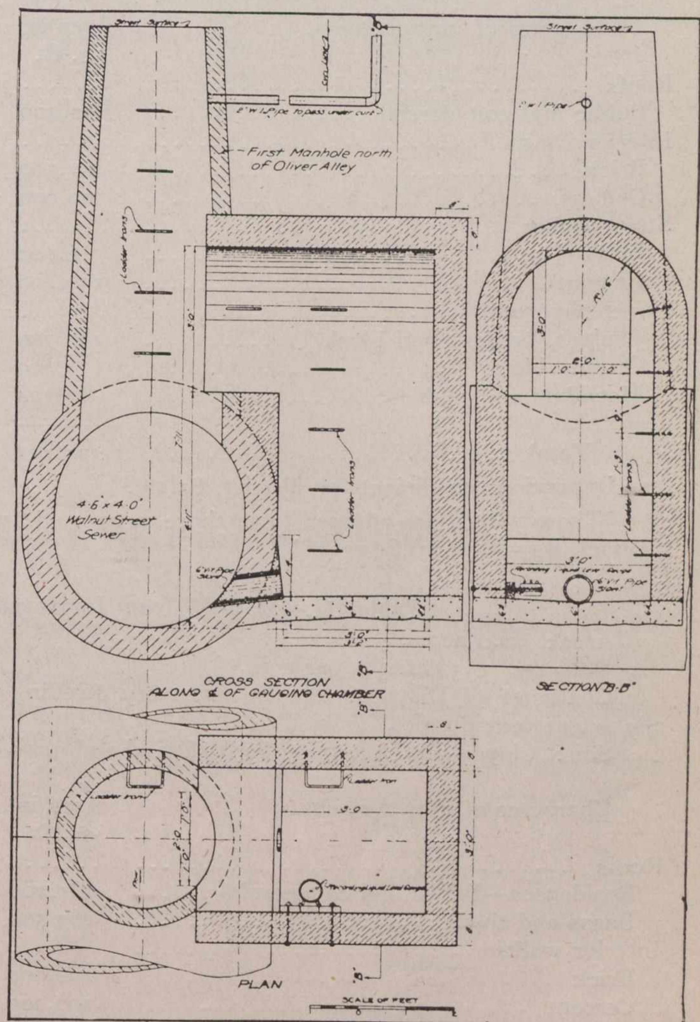


Fig. 4.—Details of Typical Gauging Chamber.

these surveys, directed their efforts in such a way as to secure the necessary data as soon as possible. But the fact that such data were not at once available in the various parts of the city requiring sewerage plans has been a handicap to the progress made in these relief investigations.

Procedure.—To determine the cause of cellar floodings or other defective drainage and to recommend the most suitable relief measures, necessitated an exhaustive study not only of the sewers in the immediate vicinity of the floodings but in the adjacent territory as well, inasmuch as it was often necessary to divert portions of the drainage into adjacent sewer systems. For this purpose all existing sewers and inlets were plotted on the portion of the topographic map covering the district under investigation. The district was then subdivided to such an extent that the drainage to each inlet could be determined.

The data were then tabulated in a manner which indicates at a glance at what points the existing sewer is of insufficient capacity, as well as the sizes and grades necessary for the proposed relief sewer. To supplement these studies a house-to-house canvass was then made to determine the extent of the territory affected by defective

(2) Inadequate provision for storm drainage in sewers intended to receive it.

(3) Defective construction of combined sewers.

The problem of providing relief for such conditions was, therefore, somewhat different from the design of new sewerage, inasmuch as it is obvious that the existing

Table II.—Run-Off from Improved Areas in Typical Commercial District.

Area: 30.4 acres.*

Character of District: Commercial. Area covered by office buildings, stores and other business houses. No undeveloped area.

Soil: Sand and gravel.

Character of improvement.	Area in sq. ft.	Per cent. of total area.	Estimated degree of imperviousness.	Equivalent impervious area, sq. ft.	Per cent. total area impervious.
Roofs	....	....	...	......	59.8
Public and commercial	881,200	66.5	.90	793,080	....
Interior walks	....	....	...	......	0.8
Brick	7,500	0.6	.40	3,000	....
Cement	10,000	0.7	.75	7,500	....
Street walks	....	....	...	......	8.1
Brick	6,100	0.5	.40	2,440	....
Cement	139,300	10.5	.75	104,475	....
Street pavements	....	....	...	......	16.4
Asphalt, brick, wood block	145,500	11.0	.85	123,675	....
Granite block	111,400	8.4	.75	83,550	....
Macadam and cobble	23,224	1.8	.40	9,290	....
Total	1,324,224	100.0	...	1,127,010	85.1

Impervious coefficient for district, 85.1%.

*These values are adapted from those suggested by Mr. Emil Kuichling in the Kuichling and Bryant "Report on Back Bay (Boston) Sewers."

Table III.—Run-Off from Improved Areas in Typical Tenement District.

Area: 60.0 acres.

Density: 135 persons per acre.

Character of District: Mostly two-story dwellings and stores, with paved interior walks and yards. Essentially a tenement district.

Soil: Clayey sand overlying beds of sand and gravel.

Character of improvement.	Area in sq. ft.	Per cent. of total area.	Estimated degree of imperviousness.	Equivalent impervious area, sq. ft.	Per cent. total area impervious.
Roofs	....	....	...	......	38.2
Residences—Public and commercial	981,200	37.6	.90	883,080	....
Barns and sheds	163,500	6.3	.70	114,450	....
Interior walks	....	....	...	......	6.3
Brick	220,000	8.4	.40	88,000	....
Cement	100,400	3.8	.75	75,300	....
Street walks	....	....	...	......	7.0
Brick	39,500	1.5	.40	15,800	....
Cement	222,600	8.5	.75	166,950	....
Street pavements	....	....	...	......	14.4
Asphalt, brick, wood block	393,900	15.1	.85	334,815	....
Granite block	29,000	1.1	.75	21,750	....
Macadam and cobble	46,800	1.8	.40	18,720	....
Unimproved	....	....	...	......	4.0
Unpaved yards and lawns	415,930	15.9	.25	103,982	....
Total	2,612,830	100.0	...	1,822,847	69.9

Impervious coefficient for district, 69.9%.

sewerage as well as to serve as a guide in the recommendation for the proposed relief.

The flooding of cellars by backwater from sewers, and other causes of defective sewerage have been traced to one or more of the following causes:

(1) The improper use of sanitary systems of sewers by the connection of roof leaders and street inlets.

sewers should be used to their fullest extent. In some cases sufficient relief could be obtained by replacing the local sewer with one of larger capacity, in others it was necessary to cut off and divert portions of the drainage into adjacent systems, while in some extreme cases it was found necessary either to reconstruct the sewer throughout its entire length to the outlet, or to provide an additional storm sewer. The latter alternative, however, was rarely possible, as it is generally necessary to place the proposed sewer at the same depth as the existing sewer in order to intercept the tributary or lateral sewers. Studies for relief sewers were carried to a point where it was possible to recommend the best and most economical methods of relief, as well as to prepare preliminary plans, estimates and reports.

Table IV.—Run-Off from Improved Areas in Typical Combined Tenement and Industrial Districts.

Area: 35.6 acres.
Density: 55 persons per acre.
Character of District: Tenement and industrial.
Soil: Clay and sand to gravel.

Character of improvement.	Area in sq. ft.	Per cent. of total area.	Estimated degree of imperviousness.	Equivalent impervious area, sq. ft.	Per cent. total area impervious.
Roofs	....	....	...	......	24.5
Residences	289,200	18.6	.90	260,280	....
Public and commercial	66,800	4.3	.90	60,120	....
Barns and sheds	79,200	5.1	.75	59,400	....
Interior walks	....	....	...	......	2.0
Brick	35,600	2.3	.40	14,240	....
Cement	22,600	1.5	.75	16,950	....
Street walks	....	....	...	......	5.0
Brick	48,200	3.1	.40	19,280	....
Cement	78,100	5.0	.75	58,575	....
Street pavements	....	....	...	......	6.2
Macadam and cobble	238,600	15.4	.40	95,440	....
Unimproved	....	....	...	......	6.7
Unpaved yards and lawns	692,436	44.7	.15	103,865	....
Total	1,550,736	100.0	...	688,150	44.4

Impervious coefficient for district, 44.4%.

Table V.—Run-Off from Improved Areas in Typical Residential District.

Area: 291.1 acres.
Density: 20 persons per acre.
Character of District: Residential. Well developed. Detached dwellings of two or more stories and in good condition; 2.3 acres open and undeveloped.
Soil: Yellow and blue clay, overlying beds of shale and limestone.

Character of improvement.	Area in sq. ft.	Per cent. of total area.	Estimated degree of imperviousness.	Equivalent impervious area, sq. ft.	Per cent. total area impervious.
Roofs	....	....	...	......	14.4
Residences	1,665,800	13.1	.90	1,499,220	....
Public and commercial	214,000	1.7	.90	192,600	....
Barns and sheds	171,900	1.4	.75	128,925	....
Interior walks	....	....	...	......	2.2
Brick	73,200	0.6	.40	29,280	....
Cement	334,400	2.6	.75	250,800	....
Street walks	....	....	...	......	2.9
Brick	127,000	1.0	.40	50,800	....
Cement	430,600	3.4	.75	322,950	....
Street pavements	....	....	...	......	7.0
Asphalt, brick, wood block	630,200	5.0	.85	535,670	....
Granite block	133,100	1.0	.75	99,825	....
Macadam and cobble	612,300	4.8	.40	244,920	....
Gravel and poor macadam	46,500	0.4	.20	9,300	....
Unimproved	....	....	...	......	9.4
Unpaved yards and lawns	....	....	...	......	....
Tributary to paved gutters	7,246,800	57.1	.15	1,087,020	....
Not tributary to paved gutters	1,000,955	7.9	.10	100,095	....
Total	12,686,755	100.0	...	4,551,405	35.9

Impervious coefficient for district, 35.9%.

Hickory is the strongest Canadian wood. When properly seasoned a hickory column will support a weight of 12 tons per square inch cross-section, which is considerably more than what could be borne by a pillar of cast iron or steel of the same length and weight.

The mineral production of Ontario in 1913 had a total value of $53,207,311, the largest yet recorded in any year. Of this $37,507,935 was of metallic, and $15,699,376 of non-metallic substances. The increase over the output for 1912 was $4,865,699, or more than 10 per cent.

FINISHING TEMPERATURES AND PROPERTIES OF RAILS.

The U.S. Bureau of Standards has carried on an investigation into the manufacture of steel rails. The main objects of it were to determine, from measurements taken in representative rail mills, the present American practice regarding the temperatures at which rails are rolled; to demonstrate the ease and accuracy with which such temperatures may be measured; to find out what the "shrinkage clause" in rail specifications really means; and finally to determine for rail steels some of the physical properties, particularly those of interest in manufacture and some of which, it would seem, are not sufficiently well known as yet. Among these last are the expansion, melting ranges, critical ranges and temperature distribution throughout a rail section on cooling.

Table 1—Summary of Ingot Temperatures at Blooming Mill.

Mill.	Number of ingots observed.	Average temp. C.	Average time in blooming mill, seconds.	Remarks.
A......	20	1140±10	120±25	Excessive time in pits.
	19	1082±16	121±16	In pits 1 hr. 35 min. to 2 hrs.
B......	30	1102±12	91±7	Two heats.
C......	29	1087±17	71±3	Two heats.
D......	43	1118±15	37±2.3	Seven heats.

In the spring of 1913, observations were taken of ingot and finishng temperatures of rails in four representative mills, designated as A, B, C, D. Temperatures were taken with the Holborn-Kurlbaum type of Morse optical pyrometer.

In Table 1 is given a summary of the temperatures of ingots as measured at the blooming mill during rolling for rails, and in Table 2 a summary of finishing temperatures of rails.

Table 3 illustrates with what uniformity of temperature it is possible to carry out the rolling of rails in practice. The letters refer to the position of the rail in the ingot. It is evident that a uniformity of ±10 deg. C. may be maintained.

An inspection of the tables shows that there is practical uniformity among the several mills for the rolling temperatures of ingots for steel rails, the range being from 1,080 deg. C. (1,975 deg. F.) to 1,140 deg. C. (2,085 deg. F.). With the exception of the Bessemer rails of Mill D, rolled at an average temperature of 1,047 deg. C. (1,917 deg. F.), there is no very considerable difference among the finishing temperatures of the rails as observed at the hot saws for the several mills, the range being about 880 deg. C. (1,615 deg. F.) to 990 deg. C. (1,815 deg. F.); or, in other words, the four mills all finished their rails to within 50 deg. C. of 935 deg. C. (1,715 deg. F.) on the average, excepting the Bessemer rails of Mill D. This temperature of 935 deg. C. is 270 deg. C. (520 deg. F.) above the mean value, 665 deg. C. (1,230 deg. F.) of the critical ranges of these rail steels. Concerning the distribution of temperatures within the head of a cooling rail, it is shown that the centre of the head is some 50 deg. C. (120 deg. F.) to 60 deg. C. hotter than the optical pyrometer reading at 935 deg. C.; therefore the centre of the head is finished, on the average, at about 325 deg. C. (615 deg. F.) above the critical range for 100-lb. sections.

The tables of finishing temperatures show several other facts of interest. For example, it is evident that it is a possible and easy operation to determine accurately the temperature of each rail length in succession as it arrives at the hot saw. (See Table 3.) From the measurements at Mill A, the relative temperatures of rails differing in weight of section, rolled from ingots having closely the same weights and temperatures, are shown. Thus the average of the finishing temperatures of the top rails (A and D or A and C) for 100, 90 and 75 lb. sections were, respectively, 988 deg., 976 deg. and 924 deg., and similarly for the others.

Chemical analyses and microphotographic examinations were also made and the mechanical properties determined for a number of samples of rail the rolling of which had been observed. From a comparison of these few observations, there appears to be not a sufficient degree of correlation to warrant associating very specifically any of the characteristics defined by these three methods of examination, either with the temperatures of rolling here observed or with each other.

The following thermal properties of these rail steels were determined in the laboratory: The critical range on heating is located (maximum) to within 7 deg. C. of 732 deg. C. (1,350 deg. F.) for the ten samples of O. H. and Bessemer steels examined. On cooling, the critical range lies between the limits 680 deg. C. (1,256 deg. F.) and 650 deg. C. (1,202 deg. F.). The melting or freezing range for rail steel extends from about 1,470 deg. C. (2,680 deg. F.) to nearly the melting point of iron located at 1,530 deg. C. (2,786 deg. F.).

Table 2—Summary of Rail Finishing Temperatures.

Mill.	A			B	C	D
Location of station	Hot saw			Hot saw.	Cooling bed No. 1.	Just before hot saws.
Distance last pass to station	4			5	3	1
Number of rails	120	70	80	312	171	186
Weight of rails	75	90	100	72	90	100
Type of rail	Bessemer	O. H.	O. H.	O. H.	O. H.	Bessemer and O. H.
Letters indicate location of rails in ingot.	A & D	A & D	A & C	A=957±8	A & D	Mean of A, B, C, D. Bessemer
	924±13	976±10	988±23	B=945±9	911±8	1047±8
	B & E	B & E	B & D	C=935±9	B & E	Mean of A, B, C, D. O.H.
	918±13	964±12	962±24	D=923±9	901±8	992±15
	C & F	C & F	....		C & F	
	906±15	942± 7	....	E=951±8	883±8	
				F=937±10	2nd Series	
				G=920±9	A & B	
				H=903±11	928±6	
				Mean A, B, C, D, 939±9	B & E	
					920±7	
				Mean E, F, G, H, 928±9	C & F	
					904±8	

The expansion for O. H. and Bessemer steels is not the same. Above 800 deg. C. (1,470 deg. F.) the expansion for both increases linearly with temperature and the linear coefficient per degree Centigrade has the following mean values from 0 deg. to 1,000 deg. C.; (1) For Bessemer steel (carbon .40 to .50 per cent.) = 0.0000146, (2) for open hearth steel (carbon .65 to .70 per cent.) = 0.0000156.

The average composition of the Bessemer steel was carbon = 0.40 to 0.50 and manganese = 0.76 to 0.93; of the open hearth steel, carbon = 0.66 to 0.70 and manganese = 0.66 to 0.72.

In 1909 the American Society for testing Materials limited the shrinkage allowance on 100 lb. sections to 6¾ in. in 33 ft., or to an equivalent of 1,947 deg. F. (1,064 deg. C.) for O.H. and 2,055 deg. F. (1,124 deg. C.) for Bessemer rails. This specification is still in force.

Table 3—Measurement of Temperatures of Head of Rail at Hot Saws for 72 Lb. Rails.

A	B	C	D	Mean A B C D	E	F	G	H	Mean E F G H
954	942	942	926	941	954	943	926	896	930
966	953	948	928	943	965	945	919	898	932
956	955	943	917	943	954	931	919	902	927
958	945	926	929	939	952	944	920	906	923
966	960	943	932	950	960	...	922	905	929
968	945	940	929	945	954	940	920	901	929
943	931	918	914	927	...	...	...	...	...
...	...	...	...	...	948	933	918	908	919
958	951	937	929	944	948	929	918	894	922
962	923	920	917	928	940	920	908	894	915
942	934	922	908	912	940	920	902	878	904
922	912	908	887	907	926	906	894	872	902
922	909	899	897	907	912	897	882	866	889
926	917	905	897	912	926	903	902	882	904
				NEW HEAT					
953	947	935	920	938	932	931	913	903	920
966	954	932	912	939	954	937	929	...	940
954	945	934	922	938	954	945	922	914	934
956	957	936	919	941	960	952	935	918	941
964	949	934	926	943	954	943	...	913	936
963	957	945	934	951	949	929	915	...	930
952	943	937	930	940	950	943	924	920	934
962	958	940	936	951	957	940	934	910	935
966	957	940	932	949	951	934	918	897	924
954	943	935	926	939	938	937	918	896	922
956	943	929	922	936	955	943	922	901	930
968	954	943	931	947	958	943	924	916	935
964	946	943	935	946	964	952	932	913	942
958	943	943	931	944	965	946	931	920	940
960	946	944	931	945	946	943	926	918	934
961	954	946	930	947	954	943	935	918	938
958	949	945	943	949	957	935	931	920	936
955	947	939	929	943	945	942	932	908	931
965	953	939	939	949	968	953	941	917	945
963	942	932	919	939	958	936	919	907	930
962	951	937	919	943	957	940	912	896	927
943	937	922	914	929	945	922	908	901	920
960	944	943	929	944	966	957	932	908	940
969	954	946	931	950	964	[illegible]	928	912	936
975	962	953	934	957	954	952	920	908	934
957	939	928	908	933	946	931	916	888	921
*957±	945±9	935±9	923±9	939±9	951±8	937±10	920±9	903±11	928±9

* This line represents the mean degree of temperature of above table.

A rail of 100 lb. section in cooling freely in air from a uniform temperature of 1,070 deg. C. (1,960 deg. F.) reaches its recalescence point at 670 deg. C. (1,238 deg. F.) in about 8 min. 30 sec. The maximum difference in temperature between the centre and outside of the head during this cooling is about 85 deg. C. at 1,000 deg. C. (1,832 deg. F.), drops to 55 deg. at 900 deg. C. and to 30 deg. at 800 deg. C., becoming 0 deg. again at 670 deg. C.

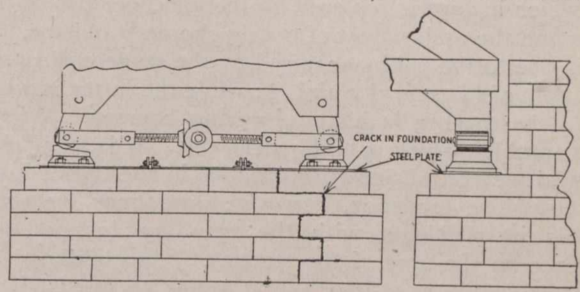
A comparison of the shrinkage clause in American rail specifications (for example, those of the A.S.T.M.) with the expansion of rail steel shows that this clause permits finishing rails at 1,120 deg. C. (2,045 deg. F.), or 450 deg. C. (840 deg. F.) above the critical range of rail steel, and above the temperature at which many ingots for rails are actually rolled in practice and well above the practice of the rail mills for finishing temperatures, as shown in Table 2. Such a shrinkage clause, therefore, does not serve the avowed purpose of limiting the finishing temperatures to a value slightly above the critical range.

In conclusion, it should be emphasized that the various series of observations recorded in this investigation are of but a preliminary nature and do not pretend to solve the question of the relations between temperature of rolling and the properties of rails. It would seem desirable to make a much more complete and comprehensive study of the various matters mentioned and of related questions than has hitherto been attempted, and on a scale commensurate with the importance to the community of the problem of sound rails.

BRIDGE REPAIR ON C.P.R.

By J. G. Koppel,
Electrical Superintendent of Bridges, Sault Ste. Marie, Ontario, Canada.

On the end piece of a swing bridge crossing the Ste. Marie Ship Canal at Soo, Ontario, a repair was recently made in a very efficient manner. The bridge is 407 feet in length, over which runs a single track for the Canadian Pacific railroad trains. The swinging and the end piece shoe-jacks operate by electric motors from a centre cabin elevated over the track. From time to time it was found that the bridge tended to tilt to one side, especially the south end, which is on a curve,



BRIDGE REPAIR ON THE CANADIAN PACIFIC.

and finally a serious crack developed in the masonry which grew worse every day, and in order to keep the bridge safe we had to devise some kind of relief or safeguard to prevent a total rupture.

Arrangements had to be made to make the necessary repairs without interrupting the running of the trains on the track, and at the same time not to interfere with boats passing through the canal. The accompanying drawing shows the crack in the masonry which amounted to fifteen-sixteenths of an inch, and also shows how the repairs were accomplished. A one-half inch steel plate was made in three sections, and drilled to fit on the jack-shoe bolts, and one jack-shoe at a time was removed and the plate laid in place and the jack-shoe replaced. When both plates were secured in position, then the centre plate was put in and the bolts screwed down, and when all were in tension the masonry was drawn into its original position, while a mixture of cement was poured into the crack, which made a very good job, as the foundation is now apparently as good as could be desired, and safer than the original structure, as nothing short of an earthquake could move the steel plate that we added to the structure.

Not only so, but it is evident at a glance that a new idea has been added to structures of this particular kind. Masonry under direct vertical pressure is always reliable, but under slightly horizontal pressure of an intermittent kind, such as is caused by the oscillation of a heavy locomotive and attached train rounding a curve the tendency of the masonry to crack or dislocate is very great, and a reinforcing plate should be used.

POSSIBLE FAULTS IN OUR METHODS OF TEACHING.

By I. F. Morrison,
Lecturer in Structural Engineering, University of Alberta.

IT is the chief function of any engineer, young or old, to solve problems, and to obtain, not only a solution, but the best solution considered from all points of view. Such a best solution requires a sound knowledge of the fundamental principles of the physical, chemical and mathematical sciences and some understanding of business methods, economics, law and many other important fields. At present it is not possible to deal with these latter requirements as extensively as seems desirable in our engineering courses within the limited amount of time allowed. The young engineer must pick these up for himself; and there is no reason why he should not do so if he has had the proper training in other fields and has learned to study effectively. The fundamental principles and their application to problems, coupled with a training in actual methods of study, seem to be the important qualities to be desired in an engineering graduate.

One of the outstanding faults of the present-day engineering student is his dependence on problems and on text-books. Such dependence is a habit due to poor training, which in turn is fostered by the improper use of text-books and the assignment of poorly chosen problems. The first is due to the habits acquired by the student during an early part of his school training; while the latter is wholly due to the teacher in his engineering courses. Without doubt, problem-work is highly desirable. It is an efficient, practical method of applying practice in the use of fundamentals. The tendency, however, is to bury the fundamentals in problems with the resulting loss of independence on the part of the student. The childhood propensity to imitate and to follow seems to persist in the minds of many engineering students even until the time of graduation. Problem-work should retard rather than promote such a growing dependence and can be used effectively in overcoming the difficulty.

No doubt the modern methods of our public school education systems are in part responsible for this condition; and for that reason alone, an inimical influence should be brought to bear upon the student by the engineering college tending towards independency of thought and ability. For this, quality and not quantity in most cases should be required in problem-work. Drill is for those who are followers, not for leaders. The engineering college should aim to effect the transition of its newly accepted students from dependent to independent thinkers, from followers to leaders with intuition and initiative.

A problem which requires an indirect solution is of greater value than several which may be quickly solved by a formula. The following problem is taken from one of the most recent text-books on applied mechanics, and is preceded in that book by an illustrative problem in which the statement is made "Apply the formula," referring to the well-known formula $F = Ma$. A force of four pounds causes a certain mass to move from rest through 18 feet in 3 seconds. Find the mass. According to the worked example which precedes it, the student applies the formula; which is a very common type and is applied in quantity. Eventually the student learns, not that the force produces an acceleration inversely proportional to the mass or an equivalent fact, but that simply $F = Ma$.

Consider the other type of problem which demands something more than a mechanical application. An iron ball, starting from rest, rolls down an inclined plane 10.2 feet long in 4.8 seconds. What is the inclination of the plane with the vertical? The solution is indirect and requires a careful consideration of the forces which act to cause the ball to roll down the plane.

The first problem, while not quite direct, demands but little reasoning. At this point the student has already learned the formula $s = \frac{1}{2}at^2$, and is now also dependent on the fact that $F = Ma$ is the ultimate solution of his problem. Such an exercise is nearly worthless, unless the student needs practice in multiplication. Many problems of this sort will make the student dependent on a formula for the solution of any problem so that ultimately his ability to solve problems will be limited to doing those which he has seen done elsewhere, or for which he can find similar cases already worked in his text-book.

To be of the greatest value, problems should demand the applications of principles and common sense rather than formulæ. They should be interesting and practical. The ability to understand new problems and to obtain the best solution for each is the ultimate aim. This can be accomplished only by independency in thought and ability.

Objections have been raised to the present method of usage of symbolic formulæ. Everyone seems to agree that such formulæ should be thought of in terms of English words and that they should be so taught; but few seem to realize that the problem-work is perhaps partly responsible for the objectionable conditions. Most students naturally follow the line of least resistance where it is made evident. "Apply the formula." A short symbolic formula is not only attractive from this point of view but is soon recognized as an easy means to an end. Problems which give merely a drill in the use of symbolic formulæ and therefore require only a mechanical operation, might well be avoided. The benefits derived from them cannot be greater than the effort applied to them

Symbolic formulæ have their place: their role is important, but their importance is secondary. We should hardly think of teaching shorthand to children who have not yet learned to read and write in the ordinary way. Much less would we think of mixing reading, writing and shorthand together, when taught. Then, why should we seek to start a subject in elementary mechanics with notations immediately followed with formulæ? Perhaps it would be well to avoid formulæ entirely for a while and to allow the use of them to be developed naturally by the student. It would be interesting to watch the development of symbols and formulæ in an elementary class. The writer believes that each student would eventually evolve some system of notation and make up his own symbolic formulæ. This would be putting the formulæ in their proper relation to the subject: it would give the proper weight to their importance.

However, the ability to translate a derived formula is of great importance. This point must not be overlooked; and the translation of formulæ which have been derived should be insisted upon. Students will see the value of writing the statement of some physical law with single letters more readily than they will observe the complete meaning of a derived formula. Nevertheless, if they have been the promoters of their own formulæ originally, this difficulty should not persist.

Efficiency, like economy, is sometimes false. Both must be developed along natural lines. It is not economy to set an unskilled workman to work with a valuable machine; nor is it efficiency to require an untrained student to use a symbolic formula. After all, formulæ are merely the result of a natural method developed to obtain efficacy.

STEEL TOWER TRANSMISSION LINES

A STUDY ON THE LOCATION AND CONSTRUCTION OF HYDRO-ELECTRIC POWER TRANSMISSION LINES AND DISCUSSION OF PRESENT EXPERIMENTS BEING CONDUCTED RELATIVE TO THEIR OPERATION.

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At first glance it might seem that transmission line location and construction falls within the field of the electrical engineer. However, it is likely that the civil engineer is a more successful locator and construction man, while a combination of electrical and civil engineering experience are the ideal requirements for a new and growing branch, soon to become known as "Transmission Engineering."

Hydro-electric power development may be classified into three operations: Generation, Transmission and Distribution.

Of the three divisions, transmission is the weakest link, and is receiving an unusual amount of study and experiment at this time.

Large water-powers are frequently located in isolated districts many miles from the power market. Scarcely two decades past, many large water-powers were not available for commercial purposes, chiefly because this art of transmission was as yet not sufficiently perfected.

Transformers were in their embryo stage and direct current almost universally used.

With the coming of modern transformers and alternating current, most water-powers were brought within commercial operating distances of a possible market.

Electrical designers have been successful in the development of reliable generators and transformers, while the solving of low-tension distribution problems has kept the pace, but the transmission link still gives cause for grave concern.

Continuity of power is absolutely essential to obtain and hold the business and good-will of the consumer, and severe penalty clauses in power contracts for interruption are the rule rather than exception.

Only a few years ago voltages of 11,000 were the average, while the more daring engineers were attempting transmission at 44,000 volts. To-day many systems are operating at pressures between 130,000 and 150,000 volts.

The location and construction of lines carrying these lower voltages were comparatively simple matters. The distances covered were short, and conductors, insulators and poles were cheap in comparison with the modern high-voltage steel tower construction.

Franchises along public highways were easily obtained, and when found advisable to cross privately owned property, the right to set a few poles could be secured at a nominal cost. The securing of a profile was needless, and the construction foreman usually chose the pole locations by inspection, while little or no attention was paid to elimination of angles.

With the rapid advance in the art of transmission, the steel tower has displaced the wood pole for high voltages. One good example of this quick change occurred under the writer's observation. An expensive wood pole line carrying a single 60,000-volt circuit, which was completed for service a little over four years ago, was pulled down during the past summer season and replaced with a double-circuit 110,000-volt steel tower line. The rule is, however, to retain the old lines and, if desirable, parallel them with a new line, the old one being kept for emergencies in case of break in the new line.

Lines carrying moderate loads, together with line pressures limited to 60,000 volts, are still supported upon poles. Pin-type insulators are still used, but the suspension insulator is very rapidly displacing the old type, even at these moderate voltages.

Steel tower lines are of two distinct types:—

(1) The rigid towers, capable of resisting horizontal stresses in all directions.

(2) The flexible type, whose function is to support the conductor only and will not resist horizontal stresses parallel to the direction of the line.

If a flexible type is used, it is necessary to specify that a rigid tower be placed about every mile to take the strains in conductor after sagging.

As the flexible tower is not commonly used, we will refer only to the rigid or square-base tower hereafter.

A modern tower line, carrying current at pressures between 130,000 and 150,000 volts, will cost from $10,000 to $14,000 per mile, and its location demands as careful study as the location of a railroad. It is an old adage that successful railroad locators are born and not made, and it seems probable that this term will soon apply to transmission locators. The characteristics necessary for a successful locator are: (1) A thorough knowledge of the factors which affect the first cost as well as subsequent operation. (2) A keen sense of observation so that physical facts are quickly noted and all advantages taken.

The essential difference between railroad and transmission line location is the elimination of a necessary limiting grade. Public highways, villages and telephone lines are to be avoided, but primarily a minimum distance between terminals is the goal sought by all locators.

A minimum number of angles is also desired, as every angle means a heavier tower with a "dead-end" and "strain-point," which are expensive.

When the pressure on an overhead transmission system exceeds a certain critical value, depending upon the spacing, diameter and elevation of the wires, there will appear on the surface of the conductors a halo-like glow, to which the name of "corona" has been given. Apart from its luminous effect, the appearance of the corona is accompanied by a certain loss of power, which becomes serious very quickly after the critical point is passed. If the electrical designer has specified wire and spacing which is near the corona limit at the average elevation of the line the loss rapidly increases with the elevation and is proportional to the length of the line passing over the high elevations.

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There are other phenomena due to corona which are purely electrical, which are of no interest to the locator, but lie within the sphere of the electrical designer.

The effect of altitude being so important, transmission lines which pass over mountain ranges must be kept in as low passes as possible, but if high points are unavoidable, the length of line over these points should be kept a minimum.

Tower lines are cheaper if located across rolling hills and valleys, where longer spans and fewer towers per mile are obtainable than in level country.

Where reliable U.S.G.S. sheets are not available, it is preferable to first run a preliminary line, then project the location. When the located line is being staked out, a careful classification of the ground crossed should be taken. This should include swamps, rock out-crops, class of timber, cultivated ground, etc.

At the same time a line of accurate levels should be run which will show, along the centre line, all breaks in ground of a foot or more. All transverse slopes of more than 10° should be recorded.

As soon as the detailed profile has been completed, tower locations can be made in the office. The method of locating towers which has proven to be the most practical is the one described in the "Electrical World" under date of September 11th, 1911, by Mr. J. M. Viehe.

On all high-voltage lines which are supported by suspension type insulators, it is necessary to carefully compute the relation between length of span and corresponding sag between the range of temperature which may be expected to occur in the district through which the line is passing.

During extremely cold weather, an uplift upon a suspension insulator will cut down the clearance between conductor and the towers which might cause "flashovers," and if the uplift is serious, would cause mechanical failure of the crossarm.

All towers of the rigid type are ordinarily designed to withstand great strain in a horizontal direction, but are not supposed to resist any great amount of upward pull. It is, therefore, necessary to place the towers in such a position that no great upward pull will be exerted.

The office procedure for properly locating towers is first to make certain assumptions for loading, i.e., assume a maximum wind pressure added to the weight of the conductor itself; then to the sum of these two must be added the weight of an arbitrary thickness of sleet which may be expected to accumulate upon the conductor at times. Several conditions are investigated for sag and tension, as sleet and ice, with a sudden drop in temperature, or a combination of any of these assumptions.

Assuming that the copper conductor will take the form of a catenary between supports, the sag for any length of span can be readily computed. A certain minimum clearance from the ground is specified, depending upon the character of country through which the line is passing. A templet representing conditions at maximum temperature is then prepared, using the same scale as the plotted profile.

After the adjacent tower locations have been made, it is necessary to test these locations for cold weather conditions. This test is applied by the use of a similar templet, which is drawn to scale, and shows the position of conductors as they will exist at minimum temperatures If the contraction of the copper is sufficient to lift the conductor above the intervening tower, it will be necessary to change the length of span on either side until a position is found where no uplift to insulators will occur and at the same time keep within the minimum prescribed clearance during maximum temperatures.

A great number of combinations will occur while studying the proper location for transmission line towers, and the most economical points for locations are often the result of several days' study.

Owing to the fact that interruption to service are not unusual from line troubles, it is best to build duplicate lines if feasible to finance the construction. If a duplicate line is authorized, it is better to construct it upon separate supports located over a different route in order to avoid the chance of destructive storms putting both lines out of commission at the same time. Separate routes for transmission lines, will, of course, increase the cost of patrolling and right-of-way, which is in turn offset by more continuous service.

All terminals of a modern power distribution system must be kept in constant telephone communication. In fact, the telephone is as necessary to the modern power system as is the telegraph and block signal to the operation of a modern railroad. Statistics show that interruptions and troubles with the average operating telephone system are more frequent and cause more worry to the operating superintendent than does the main transmission line. It frequently happens that when a break in the transmission line occurs, the telephone line will also be out of commission. This is more likely to occur if the telephone line is strung upon the same supports as the main conductor. It is, therefore, advisable to build a separate telephone line of modern construction at a sufficient distance from the transmission line to avoid the chance of crossing with the main conductors.

Five years ago transmission line towers were averaging from 2,000 to 3,000 lbs. each, while to-day towers of 5,000, 7,000 and 12,000 lbs. in weight are not infrequent. The average heavy tower when erected will cost from $700 to $1,000, and it is, therefore, good business to study the line very carefully, indeed, before finally specifying locations.

As soon as all tower locations have been made, a field party should stake these towers out upon the ground, and this is where the best of judgment must be exercised. It is frequently found that the locations as selected in the office will place the tower directly upon a pile of rocks, or in wet ground, etc. By moving the tower a short distance in either direction, these natural difficulties may be avoided. At the same time, it must be borne in mind that a slight change in tower locations will very materially alter the position of the conductors with reference to the ground, as the amount of sag changes rapidly with changes in span length. The field man must, therefore, be thoroughly acquainted with the technical detail of the scheme by which towers are located, and must be equipped with a celluloid templet in the field and adjust the small changes in location so that all requirements are fulfilled.

If there are several different types of towers, which may be used, such as different heights, weights and side-hill extensions, etc., it is possible for the field men to change the office selection of a tower with a 10 or 20 ft extension to one with a 4 ft., 6 ft. or 8 ft. side-hill extension, thereby saving expensive excavation and at the same time reducing the amount of steel.

The location of all towers having been made, the line is now ready for construction. The procedure of construction is as follows:—

(1) Clearing line of all timber, including "Danger" timber outside of the right-of-way proper. (2) Excavation for footings. (3) Distribution of stubs and tower steel. (4) Placing of stubs in concrete or earth and backfilling

same. (5) Assembling towers, including hanging of insulators or crossarms and distribution of small hardware for stringing purposes. (6) Erection of towers. (7) Stringing of conductor. (8) Cleaning-up crew.

In order that the construction men may be supplied with the best information possible, it has been found economical to condense all the information regarding location, types of towers, numbers of insulators, etc., into a pamphlet, which is furnished to each general foreman. It is also found of great assistance to the man in charge of material distribution, to be furnished with a sheet showing the railroad stations from which material must be hauled, and which will include the number and types of towers shipped to each station.

Whenever the supporting ground is poor or ground water is known to stay continuously in the holes which are excavated for stubs, it is always good policy to concrete the stubs and protect the channels which connect to the towers with a concrete collar, but if the ground is good, firm, dry earth, concreting is not necessary except for angle and dead-end towers. If the line to be constructed specifies towers which range from 3,000 to 3,500 lbs. it is cheaper to leave the holes open and bolt the stubs to the towers after erection, no templet being necessary to hold the stubs in place. On the other hand, with heavy towers, ranging from 5,000 to 12,000 lbs. and upward, it is better to set and backfill all stubs previous to the erection of towers by means of a steel or wood templet.

Common labor is all that is necessary for the entire construction program, including stringing of conductor, but it has been noted that within the last ten years labor unions have been insisting that union steel workers shall be specified for assembling and erection, and that union linemen must be used for stringing conductor. This is entirely unnecessary as far as results of the work are concerned, as it only takes a few weeks to train men to assemble and erect towers, and string conductor. In fact, it has been found that skilled steel workers and linemen have to be trained to this class of work in the same manner as common labor.

There are several methods of erection of towers, but the writer prefers the "A" frame method. A crew of 8 men will assemble on the average of from 2 to 3 towers per day and a crew of 15 men will erect from 8 to 16 towers daily if the ground is not too rough.

The conductor used upon modern heavy transmission lines will run from 150,000 to 300,000 c.m.; 250,000 c.m. conductor weighs about ¾ lb. per lineal foot. The tackle and grips necessary to pull this conductor must be unusually heavy. The conductor comes from the manufacturer in ½ mile or mile reels, weighing from 4,000 to 6,000 lbs each, and these reels are usually mounted upon some simple standard. An eccentric reel has been found very convenient. The conductor is pulled out by teams and passed through especially designed wooden snatch-blocks, the sheave for which is preferably lignum vitae.

The splice between two pieces of heavy conductor must be very carefully made. Copper splicing sleeves are now universally used with which to twist them into position.

It is very necessary to protect hard-drawn copper from being injured at the surface, and every precaution is taken to keep the conductor from being scored by rocks or being struck by any object which will crack the surface. The conductor is usually sagged and tied in at every mile. The pull necessary to properly sag the conductor depends upon the temperature. This stress is measured by the use of a dynamometer, but it is preferable to check the sag by sighting from one tower to the next to avoid trouble in case the dynamometer should get out of repair.

An average crew of 12 men will string from one to two miles of conductor per day, consisting of 6 copper wires and 2⅜ inch steel ground wires. The entire organization necessary to construct a line 135 miles long of 250,000 c.m. conductor during a period of seven months, is approximately as follows:—

1—Superintendent
1—Assistant Superintendent
2—General Foreman
1—Cost Clerk
1—Cashier

2—Clerks
2—Stenographer
1—Head Timekeeper
8—Assistant Timekeepers
1—Chief Material Man

2—Clearing Crews,	1—Foreman and	20 men (each crew)
2—Hole Digging Crews	1 " "	40 men " "
2—Distributing tower steel, concrete, material, conductor, etc.	1 " "	4 helpers and 50 teams (each)
3—Concrete Crews,	1 " "	20 men (each)
3—Assembling Crews,	1 " "	4 straw bosses and 40 men (each)
3—Tower Erection Crews,	1 " "	18 men and 2 teams (each)
2—Wire Stringing Crews,	1 " "	50 men and 8 teams (each)
2—Clean-up Crews	1 " "	20 men (each)

Commissary and Quartermaster Equipment.

8—Camps each equipped with an average of 20 tents each.
1—Cook
2—Flunkies
1—Commissary Clerk
NOTE: All crews are not working at one time
TOTAL AVERAGE FORCE—600 Men
100 Teams

The advance in the art of transmission has been great in the last ten years, and as yet no stable conditions seem to have arrived. It, therefore, seems doubtful to the writer that it is advisable at this time to spend so much capital in constructing heavy steel tower lines which may be obsolete within six or eight years. The life of modern towers, if properly erected and inspected, will average from 30 to 50 years. We believe that until the present rapid advance in the art of transmission has more nearly reached a standard, it would be better to construct with the idea in view of re-building in ten years. On the contrary, it is claimed that the investing public would not be interested in financing temporary construction, but will cheerfully invest in projects calling for permanent construction.

The length of span between towers is usually not limited by the strength of the conductor itself, but rather by the horizontal spacing necessary to avoid crossing due to side-swaying. Whenever spans of unusual length are necessary, copper-clad steel conductor is specified. This has practically all of the strength of a high-grade steel cable, coupled with the conductivity of copper. By pulling this conductor to a much higher tension than ordinary hard-drawn copper, sufficient horizontal clearance between conductors can be secured and still retain towers using standard clearance.

During the past year or so, some transmission engineers are attacking the policy of increasing the weights of towers and extending the lengths of spans, claiming this policy to be false economy. At a recent convention of the American Institute of Electrical Engineers, held at Vancouver, B.C., a very spirited discussion took place over the question as to whether it would not be better to reduce the length of spans rather than increase them.

The Provincial Government of British Columbia is erecting two new pumping stations on the dyked lands and adjoining the Pitt River. One of these stations is to be located on the Maple Ridge side and the other on the lower Coquitlam side of the river. These new plants have been necessitated by the growing utilization of these dyke lands. In the last two years a number of industries have been located on the Coquitlam side, and these required considerable land drainage pumping during the summer freshet season.

SURGE TANK PROBLEMS V.

CONTINUATION OF ANALYSIS OF SPILLWAY EFFECT — STUDY OF SURGE TANK WITH VARIABLE CROSS-SECTION — APPROXIMATE FORMULA FOR SURGE TANKS WITH CONSTANT CROSS-SECTION.

By PROF. FRANZ PRASIL.

Authorized Translation by E. R. Weinmann and D. R. Cooper, Hydraulic Engineers, New York City.

Investigation of the Influence of $h = u.v^2$.

Finally, a study may be added which permits a comparison between the results of the assumption already made (that $h = n.v$), and the results obtained if we use the usual formula $h = u.v^2 = u^1.\frac{Lv^2}{P.2g}$ where the friction coefficient u^1 is again assumed as constant (P = hydraulic radius of the conduit). With this condition it is necessary to use the first original equation (15) in which we must, according to the direction of the flow in the conduit, be careful to use the right sign for h. If this makes the investigation somewhat complicated, it may be said that the differential equation, which we obtain introducing the value of v in the continuity equation (17) is not only of the second (i.e., higher) order, but also of the second degree, so that difficulties occur in the determination of the general integral. This is especially the case if in the equation mentioned $q > o$ and variable.

Therefore, in the following we consider only for comparative purposes the case of a sudden shut-down of Q and for the phase of the first surge. In order to get more generality, we first consider the surge tank section as variable with the height, which means that it is assumed as a given function of z. The motion equation is then:

$$\frac{L\,dv}{g\,dt} + z + h = o. \qquad - \quad - \qquad (15\ A).$$

The equation of continuity is:

$$a.v = A.s \qquad - \quad - \quad - \qquad (17\ B)$$

If we multiply equation 15 A with $a.v.dt$, it follows, considering equation (17 B)

$$\frac{L.a}{g} v.dv + zA\,s\,dt + h.A.s.dt = o \qquad (90)$$

$A.s\ dt = A.dz = dV$ represents the change of the water volume V in the time dt. It also represents, with given surge tank dimensions as function of z or with reference to the graphical demonstration appearing further on, the function of a variable x where $z = x - h_1$ and where V is measured positive up from that level which lies below $n - n$. That is, for $z = -h_1$ or $x = o$; $V = o$.

With $h = \frac{u^1\,L\,v^2}{P.2g}$ and $h_1 = \frac{u^1\,L\,v_2^2}{P.2g}$ and multiplying (90) with $\frac{2\,g}{L\,a}$ we get

$$d(v)^2 + [\frac{2\,g}{L.a} . x + \frac{u^1}{P.a}(v^2 - v_2^2)]\, dV = o$$

If we say $v^2 = y$ $\quad v_2^2 = y_2 \quad \frac{2\,g}{L.a} = a_1 \quad \frac{u^1}{P.a} = \frac{1}{a_2}$ the total differential equation follows:

$$dy + [a_1\,x + \frac{1}{a_2}(y - y_2)]\, dV = o$$ which integration can be done introducing the integrating factor $\mu = e^{\frac{V}{a_2}}$

$$y - y_2 = -a_1\,e^{-\frac{V}{a_2}} \int_0^V x\,e^{\frac{V}{a_2}}\, dV$$ and introducing the values for y, a_1, a_2.

$$v^2 = v_2^2 - \frac{2\,g}{L.a}\,e^{-\frac{u^1 V}{P.a}} \int_0^V x\,e^{+\frac{u^1\,V}{P.a}} . dV \qquad (91)$$

With help of this equation v may be found as a function of V and x, because by given relatión of the volume V to x and vice versâ, in formula (91)

$$\int_0^V x\,e^{+\frac{u^1\,V}{P.a}} .dV$$ may be determined by means of quadrature even if $V = F\,(x)$ is given only by a curve. The method is illustrated by the diagram shown in Fig. 12. The relation of the volume V to x is shown by a curve, the abscissæ of which represent the values of V and the ordinates the values of x. The construction of the x curve $I - I$ may be accomplished, based on the given dimensions of the surge tank. $V = o$ corresponds, as said before, to the operating level for a flow of Q_1 cubic feet per second. With the same abscissæ, we construct a curve II — II, the ordinates of which are given by the values $e^{+\frac{u^1\,V}{P.a}}$ and another curve III — III with the ordinates $x.e^{\frac{u^1 V}{P.a}}$ and then the integral curve IV — IV. If we construct a further curve V — V, the ordinates of which we get if we multiply the ordinate values of the integral curve by $\frac{2\,g}{L.a}\,e^{\frac{u^1\,V}{P.a}}$ and if we construct at the distance (v_2^2) a

line parallel to the axis of abscissæ we then see that the difference of the ordinates between the (v_2^2) line and the curve V — V gives the values of v^2. Naturally, we must select suitable scales for all curves, that is to say, for their ordinates. The scale for the last curve is naturally the same as for (v_2^2). The intersection of the last curve with the parallel (v_2^2) determines the ordinate which gives the value of V on the axis of abscissæ, for which $v = o$, that is, when the highest water elevation occurs. The corresponding ordinate of the x curve gives the height of the highest elevation above the operating level.

If A is constant: $V = A.x$ and equation 91 becomes

$$v^2 = v_2^2 - \frac{2\,g.A}{L.a} . e^{-u^1 \frac{A.x}{P.a}} \int_0^x x\, e^{u^1 \frac{A\,x}{P.a}} dx$$

If we introduce $\frac{2\,g}{L} . \frac{A}{a} = \frac{1}{B^2}$; $u^1 \frac{A}{P.a} = \frac{1}{D}$ we get

$$\frac{1}{B^2} = \frac{2\,g\,A}{L.a} = \frac{1}{2.094} \qquad B = 1.447 \text{ sec.}$$

$$\frac{v_2^2\, B^2}{D^2} + 1 = \left(\frac{6.63^2 \times 1.447^2}{9.6^2}\right) + 1 = 2.00$$

Therefore

$$\frac{X}{D} + e^{-\frac{X}{D}} = 2.00$$

$$\frac{X}{D} = 1.86 \qquad X = 17.81 \text{ feet}$$

In case (A), where we assume the same conditions with $h_1 = n.v$ in which $n = 1.445$ seconds, we got z max $= 6.55'$ and therefore the highest elevation above the initial level z max $+ h_1 = 6.55 + 9.6 = 16.15'$. This

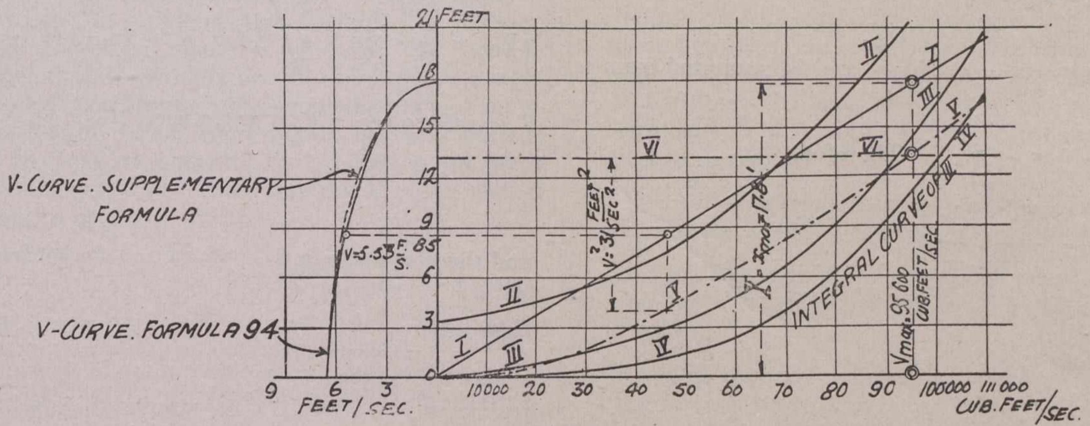

Fig. 12.

$$v^2 = v_2^2 - \frac{e^{-\frac{x}{D}}}{B^2} \int_0^x x\, e^{\frac{x}{D}} .dx$$

$$v^2 = v_2^2 + \frac{D^2}{B^2}\left(1 - \frac{x}{D} - e^{-\frac{x}{D}}\right) \qquad (92)$$

And for the computation of the maximum value X of x ($v = o$)

$$\frac{v_2^2 . B^2}{D^2} + 1 = \frac{X}{D} + e^{-\frac{X}{D}} \qquad (93)$$

The results here obtained may now be used for comparative purposes with the results of the example of case (A).

$v_2 = 6.63$ feet per sec. $\qquad \frac{v_2^2}{2\,g} = .68$ feet

$h_1 = 9.6$ feet $= \frac{u^1}{P} . 9050.0.68$

$\frac{u^1}{P} = \frac{1}{641} \qquad \frac{1}{D} = \frac{u^1}{P} . \frac{A}{a} = \frac{1}{9.6}$

value is smaller than the value which we computed for X by 1.66 feet, and it is necessary for us to correct the formulæ obtained under the assumption that $h = n.v$, in order to get as near as possible to the new results. For this purpose, we assume that for the expression of the elevation x above the initial level the formula

$$\left.\begin{aligned} x &= h_1 + z = h_1 + R.e^{-\frac{t}{2\,T_0}} \sin\left(\beta + \frac{t}{T_1}\right) \\ \frac{dx}{dt} &= s = \frac{R}{T} . e^{-\frac{t}{2\,T_0}} \sin\left(2\gamma - \beta - \frac{t}{T_1}\right) \end{aligned}\right\} \qquad (94)$$

with $tg\,\gamma = \frac{2\,T_0}{T_1}$

and

$$\frac{1}{T^2} = \frac{1}{T_1^2} + \frac{1}{(2\,T_0)^2}$$

and therefore

$$\frac{d^2x}{dt^2} = -\frac{R}{T^2}\, e^{-\frac{t}{2\,T_0}} \sin\left(2\gamma - \beta - \frac{t}{T_1}\right) \qquad (95)$$

should be used, where the values R and β are again to be determined from the initial conditions, that is to say, where $t = o$, $x = o$ and $z = h_1$, $\frac{dx}{dt} = c_1$, but the values

T_0 and T and T_1 must be determined, not as before from the given dimension, but from the condition that X becomes the maximum value of the elevation above the initial elevation, that is, for $x = X$ the derivative $\frac{dx}{dt}$ must become zero, and as before for $x = o$, the second derivative $\frac{d^2x}{dt^2}$ becomes zero. From the latter condition follows:

$2\gamma - \beta = 180^\circ = \pi$ therefore $\gamma - \beta = 180 - \gamma$
$\beta = 2\gamma - 180^\circ$ and for $t = o$ we get

$$o = h_1 - R \cdot \sin 2\gamma$$

$$c_1 = \frac{R}{2T_0} \sin\gamma \sqrt{\left(\frac{2T_0}{T_1}\right)^2 + 1}$$

because $\frac{1}{T} = \frac{1}{2T_0}\sqrt{\left(\frac{2T_0}{T_1}\right)^2 + 1}$. See equation (38).

The time t_x when $x = x_{max} = X$ $\left(\frac{dx}{dt} = o\right)$ is computed from

$\frac{t_x}{T_1} = \gamma - \beta = \pi - \gamma.$ Therefore

$$X = h_1 + R e^{-\frac{T_1}{2T_0} \cdot \frac{t_x}{T_1}} \sin(\beta + \gamma - \beta)$$

and, as $\frac{T_1}{2T_0} = ctg\,\gamma$

$$X = h_1 + R e^{-(\pi - \gamma)\, ctg\,\gamma} \cdot \sin\gamma \qquad (96)$$

and we can first determine γ with

$$R = \frac{h_1}{\sin 2\gamma}; \quad \cos\gamma \cdot e^{+(\pi - \gamma)\, cotg\,\gamma} = \frac{1}{2\left(\frac{X}{h_1} - 1\right)}$$

Is γ with this equation computed, we have for the other values

$$R = \frac{h_1}{\sin 2\gamma}; \quad 2T_0 = \frac{R}{c_1}\sqrt{tg^2\gamma + 1} = \frac{R}{C_1 . \cos\gamma};$$

$$T_1 = \frac{2T_0}{tg\,\gamma}; \quad \frac{1}{T} = \sqrt{\frac{1}{(2T_0)^2} + \frac{1}{T_1^2}}.$$

These values may then be introduced in the formulas for

$$x \text{ and } \frac{dx}{dt} = s \quad - \quad - \quad - \quad (94)$$

If the movement so determined must be similar to that which occurs if $h = u^1 \frac{L}{P} \cdot \frac{v^2}{2g}$, then the velocities $v = s \frac{A}{a}$, which we get for the same elevation x in both cases, must have the same values. If these values are not exactly equal, but if we get approximately equal values it depends on the degree of approximation desired whether we use the supplementary formula (92) or not.

In order to investigate this case, we proceed to compute the example.

With $X = 17.81$ feet $h_1 = 9.6$ feet we get

$$\cos\gamma\, e^{+(\pi - \gamma)\, cotg\,\gamma} = \frac{1}{2\left(\frac{X}{h_1} - 1\right)} = .581 \quad \gamma = 71^\circ\, 50'$$

$$\beta = 2\gamma - 180^\circ = -36^\circ\, 20'; \qquad tg\,\gamma = \frac{2T_0}{T_1} = 3.047;$$

$$R = \frac{h_1}{\sin 2\gamma} = 16.2 \text{ feet with } c_1 = 0.098 \frac{\text{feet}}{\text{sec.}}$$

$$2T_0 = \frac{R}{c_1} \sin\gamma \sqrt{tg^2\gamma + 1} = \frac{R}{c_1} tg\,\gamma = 500 \text{ sec.}$$

$T_1 = 164.3$ sec. $T = 155.9$ sec. $t_x\ (x = X) = 310$ sec.

and therefore $x = 9.6 + 16.2\, e^{-\frac{t}{500}} \sin\left(\frac{t}{164.3} - 36^\circ\right)$

$$s = \frac{16.2}{155.9} e^{-\frac{t}{500}} \sin\left(71^\circ\, 50' + \frac{t}{164.3}\right); \quad v = \frac{s.A}{a} = s \times \frac{5380}{80}$$

With these formulæ, we may compute the relative values of x, s and v for the different values of t. From the supplementary formula (92)

$$v^2 = v_2^2 + \frac{D^2}{B^2}\left(1 - \frac{x}{D} - e^{-\frac{x}{D}}\right) \quad - \quad (92)$$

we may compute the values of v for the same values of x or they may be determined graphically from Fig. 12.

They correspond to the assumption that $h = u^1 \frac{L}{P} \cdot \frac{v^2}{2g}$

and we get the following table of values:

t Sec.	o	50	100	150	200	250	.300	310
x feet	0	4.06	9.21	12 9	15.7	17.4	17.8	17.81
s feet / sec.	.098	.092	.082	.0823	.0425	.023	.00328	.00

and from that

v feet / sec.	6.63	6.25	5.49	4.23	2.92	1.51	.20	.00

But from Formula (92)

v feet / sec.	6.63	6.3	5.38	4.05	2.83	1.35	.066	.00

If we plot in the rectangular co-ordinate system (Fig. 12) the two values of v as ordinates with the values of x

as corresponding abscissæ, then we see from the curves thus obtained that we may use the supplementary formula with sufficient accuracy for the determination of the movement. If we compare the values of T_0 and T as they have been found for both assumptions of h, that is to say, for T_0 the values 194.5 and 250.0 and for T the values 137.5 and 155.9, then we see that the second value of T_0 is 1.28 times as large as the first, the second value of T is $1.13 = \sqrt{1.28}$ times as large as its first value. The first values have been found from the formulæ $T_0 = \dfrac{L}{n.g}$ and $T = \sqrt{\dfrac{L}{g} \cdot \dfrac{A}{a}}$ and from this we see the importance of the supplementary formula (92). It is evolved from the principal equation 15 if we introduce instead of L, a distance L^* which is 28% larger than L (provided that the value n is the same according to the relation $h_1 = n.v_1$) then with the distance L^*, we compute the values T and T_0. It is evident that with the introduction of the formula of resistance $h_1 = n.v$, the influence of the friction has been introduced in the equation as too large. We have, therefore, assumed in the supplementary formula the working capacity and, therefore, also the mass of the conduit volume, larger than they really are, but obtained the right value of the elevation through the supplementary formula. The friction height which corresponds to the elevation $X = 17.81$ feet can be obtained by computing the work balance, according to the former scheme, if we introduce the actual length of the conduit.

G becomes $5,380 \times 17.81 = 2,994$ tons;

$$y_1 = \tfrac{1}{2}\, 17.81 = 8.9 \text{ feet.}$$

USEFUL WORK IN FEET TONS		USED WORK	
15,800	KINETIC ENERGY OF CONDUIT VOLUME		
28,740	POTENTIAL ENERGY OF G=2994t FOR h=9·6		
	LIFTING WORK OF 2990t FOR 8·9'	26,650	TOTAL
TOTAL 44,540	FRICTION WORK	17,890	44,540

The total friction work of 17,900 foot-tons corresponds, therefore, to an average friction height with respect to the weight of the water of

$$h_{\text{average}} = \frac{17,900}{2,994} = .60\, h_1$$

For case (A) we found $h_{\text{average}} = .755\, h_1$ as the average value which corresponds to an elevation of only 16.75 feet actually. The maximum elevation above the initial level will be larger than 16.75 feet and smaller than 17.81 feet, therefore, $0.60\, h_1 < h_{\text{average}} < .755\, h_1$. The true value will be apparent if observation results in constructions of larger dimensions than can be obtained. We may remark also that the above determined values of T and T_0 in the supplementary formula are only correct for the first surge from $x = o$ to $x = X$. We may carry the mathematical investigation further and find the values for the drop from the first maximum elevation to the first minimum level. Other values for T and T_0 must be introduced, which correspond to smaller values of L^*. This repeats itself for the further phases of increase and decrease until finally L^* becomes nearly equal to L. This corresponds to the decrease of the velocity fluctuations during the movement and also with the decrease of the influence of friction.

Conclusions.

We may summarize our results as follows:

1. If the conduit ends in a pond, from which the penstock goes directly to the turbine, and if the area of the pond surface $A > 150.m.a$ where $a =$ the conduit area in square feet and m the number of miles conduit length: we need not expect any periodical fluctuations of the outflow, even if the outflow varies during the observed time. If $A < 150\ m\ a$ as is the case with artificial surge tanks, such periodical fluctuations may occur.

2. The maximum elevation above the initial level (operation level for full outflow) has approximately the same ratio for a short time for a sudden as well as for a gradual close of the whole outflow.

3. The value of the maximum elevation above the initial level may best be computed by means of the work balance, where for the average friction height 0.7 of the value of that height h_1 must be introduced, which corresponds to the drop of the water surface in the surge tank below the level of the water surface before the conduit for full outflow.

4. The extent of drop below the static level for full opening is approximately equal to the rise above computed.

5. For the determination of the oscillations during and after completed shut-down, we may use for a fair degree of approximation the formulæ, computations and demonstration methods which correspond to the analysis above developed on the theory of damped oscillations. The values of T and T_0 and therefore also T_1 may be determined if investigating the maximum elevations, as shown in our last study.

6. For operation, which results in a periodical variation of the outflow, resonance features may occur.

7. The maximum elevations may be decreased by construction of spillways in the surge tank or in the conduit.

The theory furnishes a good explanation concerning the manner and the relative value of the operations close enough for most practical purposes. But a high degree of accuracy will be accomplished only where experiments furnish much more exact values for friction in conduits under different conditions.

Appendix A—Surge Tank With Variable Cross-section.

Assuming a quadratical relation between friction-head and conduit velocity the following relation was obtained. (See Formula 91.)

$$v^2 = v_2^2 - \frac{2g}{L.a}\, e^{-\frac{u^1 V}{P.a}} \int_0^V x\, e^{+\frac{u^1 V}{P.a}}\, dV \qquad (91)$$

The maximum surge is obtained by making $v^2 = o$ and solving the latter equation for X. The previously described graphical method may be applied without further comment to surge tanks of variable cross-section with the exception that the X curve is no longer a linear function of V. For this reason and because of complex integration due to the insertion of a complicated function the graphical method is more appropriate than the analytical.

In the following example this method is applied to a surge tank, circular in section, which has a sectional area of 5,380 sq. ft. at the operating level, as was assumed before. However, in this case the sectional area increases in direct proportion to the height in such a ratio that at a height of 19.7 ft. the sectional area is 6,450 sq.

ft. (See Fig. 14.) All other data for the problem is the same as assumed in the main article.

L = 9,050 ft.
a = 80 sq. ft.
p = 32.8 ft.
Q = 530 cu. ft. per sec.
v_2 = 6.63 ft. per sec.
P = 2.44
h_1 = 9.6 ft.
u' = .0038

$$\frac{u'}{P.a} = \frac{1}{51{,}300} = \frac{1}{a_2}$$

$$6.63^2 = 44.0 = \frac{64.4}{9050.80} e^{-\frac{V}{51{,}300}} \int_0^V x e^{+\frac{V}{51{,}300}} dV$$

With this graphical method (Fig. 13) a maximum surge of only 17.0 ft. is obtained, whereas in the former example, with a surge tank of a constant sectional area

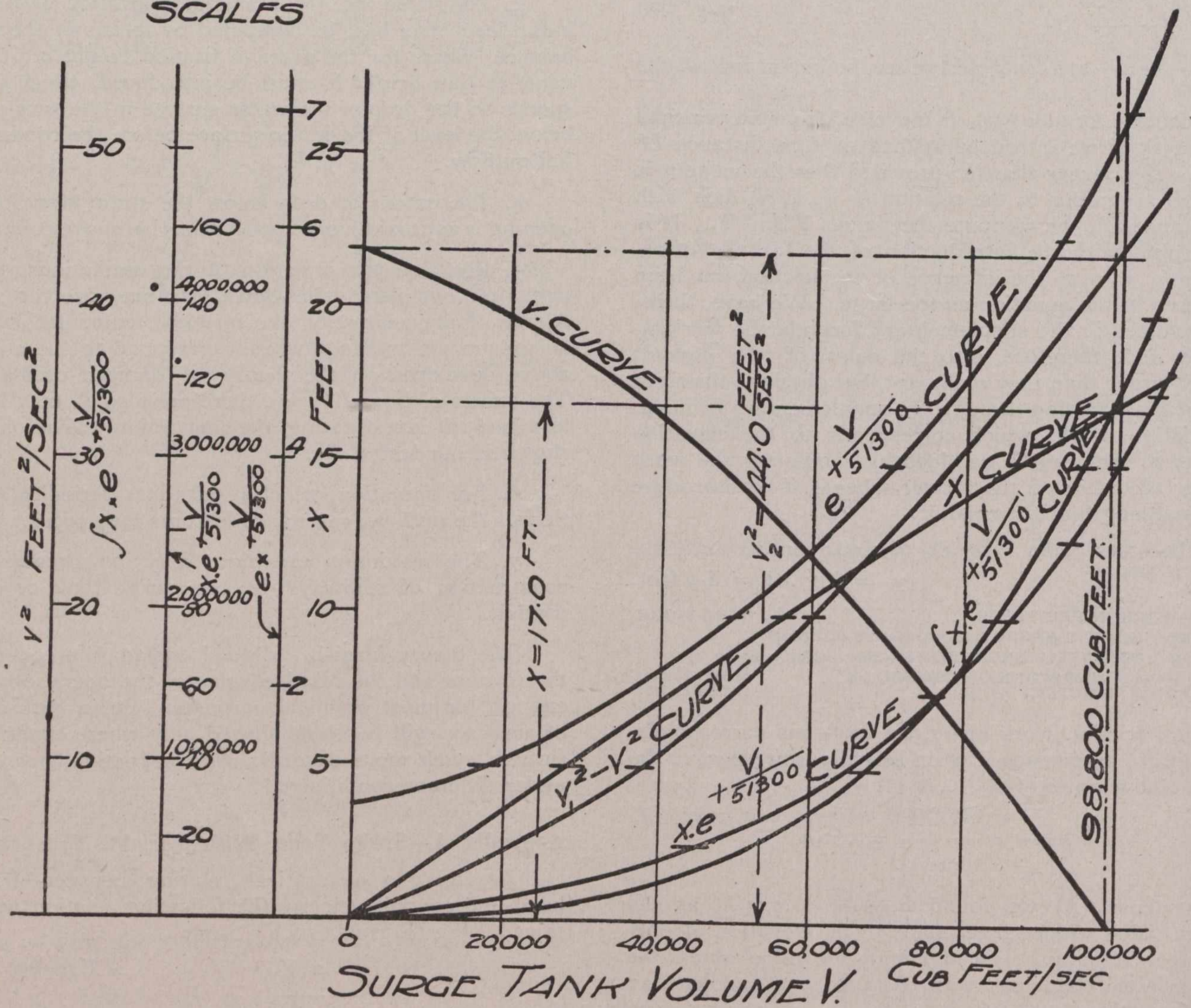

Fig. 13.

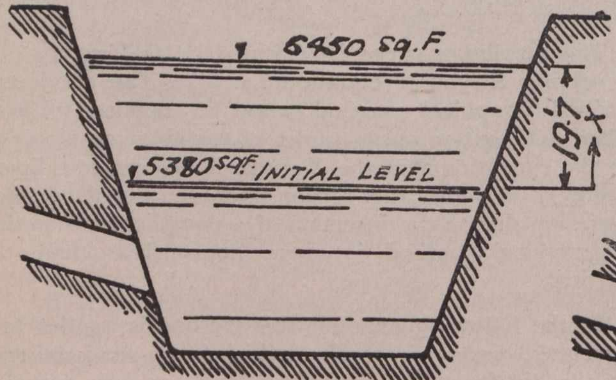

Fig. 14.

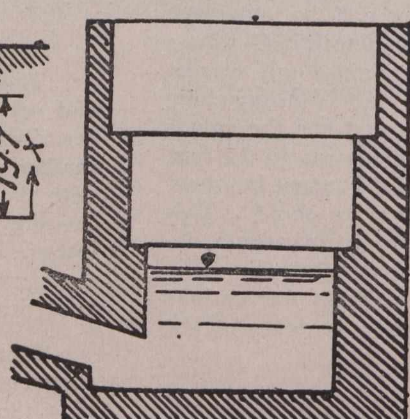

Fig. 15.

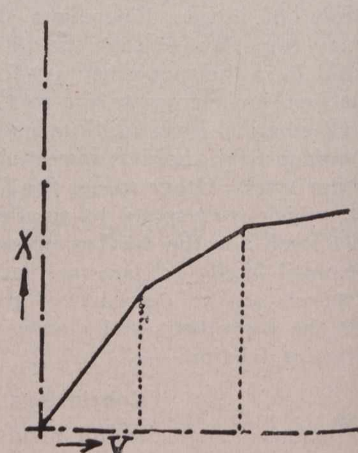

Fig. 16.

of 5,380 sq. ft. a surge of 17.8 ft. resulted. But the volume of water lifted is, in this case, somewhat greater, that is, 98,800 cu. ft., as compared to but 95,600 cu. ft. in the former case.

In the same easy way the graphical method may be applied to a surge tank in which the cross-section varies irregularly with the height. For instance, as the Albula plant of the city of Zurich. (Fig. 15.) In this a break occurs at the points on the x curve where the ratio between sectional area and height changes. (See Fig. 16.)

Appendix B—Simplified and Approximate Formula for the Determination of Surge Tanks With Constant Cross-Section.

By A. Strickler.

The designing engineer must often consider several solutions for one problem and from these select the best for the actual work. For these preliminary investigations he is in need of a simple slide-rule formula. In the following such a formula will be derived for the required area of a surge tank with given maximum surge.

As initial formula, equation 51 may be used for a sudden full shut-down.

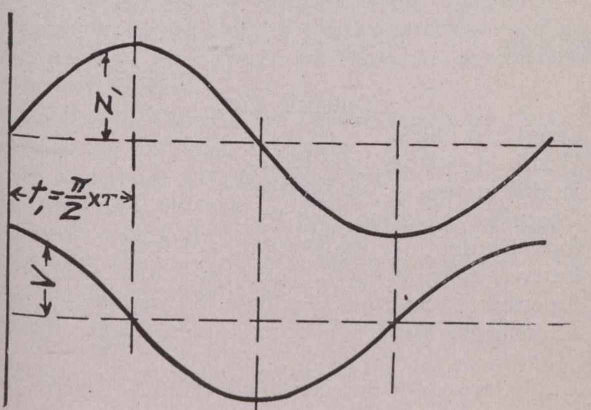

Fig. 17.

$$\frac{M \cdot v_1^2}{2} + G \cdot h_1 = G \cdot y_1 + A^1 \qquad - \qquad (51)$$

The notation being the same as before, the average friction head in the surge tank for the first surge may be called h_a. Equation 51 can now be written:

$$-\frac{w.a.L}{2g} \cdot v_1^2 + G\left(\frac{y_{max}}{2} - h_1\right) + G \cdot h_a = 0$$

$$-\frac{a.L.v_1^2}{2g} + \frac{A.y^2_{max}}{2} - A.y_{max}.h_1 + A.y_{max}.h_a = 0 \quad (51a)$$

since $h = \frac{u^1.L}{P.2g} \cdot v_1^2$; $\quad h_a = \frac{1}{t_1}\int_0^{t_1} h.dt$

$$= \frac{1}{t_1}\int_0^{t_1} \frac{u^1.L\,v_1^2}{P.2g} \cdot dt \quad (51b)$$

t_1 = time from the beginning of the surge to the moment of maximum elevation. In order to integrate equation 51b, v must be known as function of t. The simplifying assumption is now made that the first part of the v curve is a regular cosine curve. This would be true if the water level fluctuations were an undamped harmonic of the form

$z = C \cdot \sin\left(\frac{t}{T}\right)$ (see Fig. 17).

Making the above assumption

$$v = \frac{s.A}{a} = \frac{A}{a} \cdot \frac{dz}{dt} = \frac{A}{a} \cdot \frac{C}{T} \cdot \cos\left(\frac{t}{T}\right)$$

and also $\quad v_1 = \frac{A}{a} \cdot \frac{C}{T} \cdot \cos\frac{0}{T} = \frac{A}{a} \cdot \frac{C}{T}$

$$v = v_1 \cos\frac{t}{T}; \qquad h = h_1 \cdot \cos^2\left(\frac{t}{T}\right)$$

Substituting these values in equation (51b) there results:

$$\int h_a = \frac{1}{t_1}\int_0^{t_1} h_1 \cdot \cos^2\left(\frac{t}{T}\right) dt = \frac{h_1}{\frac{\pi}{2}T}\int_0^{\frac{\pi}{2}T} \cos^2\left(\frac{t}{T}\right) dt \quad (51c)$$

$$= \frac{h_1}{\frac{\pi}{2}.T} \times \frac{\pi.T}{4} = \frac{h_1}{2} \qquad \therefore \quad h_a = \frac{h_1}{2}.$$

The average friction head in the tank for the time up to the maximum surge is therefore equal, under the above mentioned assumption, to half the friction head which exists in the conduit during operation. This value of h_a introduced into equation 51a (work balance) gives:

$$-\frac{a.L}{2g} \cdot v_1^2 + \frac{A}{2} \cdot y^2_{max} - A \cdot y_{max} \cdot h_1 + \frac{A.y_{max}.h_1}{2} = 0$$

$$\text{or } A = a \cdot \frac{\frac{L}{g} \cdot v_1^2}{y_{max}\,(y_{max} - h_1)} = \frac{a\,\frac{L \cdot v_1^2}{g}}{(z_{max} + h_1)\,z_{max}}$$

$$\therefore A = \frac{a.v_1^2}{2g} \cdot \frac{2L}{(z_{max} + h_1)\,z_{max}}.$$

This formula gives results somewhat too large (1% to 3%) but is exact enough for preliminary computations.

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The surge tank at San Francisquito power station No. 1, with discussions by R. D. Johnson, Roy Taylor and W. F. Durand. Engineering Record, years 1913 and 1914.

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NOTE.

The article "Surge Tank Problems" will be issued at a later date in pamphlet form. A note will appear in *The Canadian Engineer* when ready for delivery.

PANAMA CANAL OPENED.

THE United States War Department steamship "Ancon" has made the passage through the Panama Canal, and transit through the waterway is now officially open to the traffic of the world. The "Ancon" made its way from its berth at Cristobal to the end of deep water channel from the Atlantic to the Gatun Locks. It went through the locks, which have a lift of 85 feet, in 70 minutes. It continued through the waterway, from deep water on the Atlantic to deep water on the Pacific side, without incident.

Vessels drawing not more than 30 feet of water and up to 10,000 tons register may now make the passage. It will be possible to put some of the big American dreadnoughts through at any time.

Facts About the Canal.

Total length of canal, 50 miles.
Salt water, channel to Gatun Locks, 7 miles.
Fresh water, Gatun Lake and Culebra Cut, 33 miles.
Fresh water, Pedro Miguel Lock to Miraflores Locks, 1½ miles.
Salt water, Miraflores Locks to Pacific, 8½ miles.
Width of channel, 300 to 1,000 feet.
Minimum depth, salt water, 40 feet.
Minimum depth, fresh water, 41½ feet.
Total angles in canal, 600d 51m.
Sharpest angle, Tabernilla, 67d 10m.
Work begun by Americans, May 4, 1904.
Number of men employed, average, 40,000.
Steam shovels employed, 101.
Locomotives employed, 307.
Drills employed, 4,572.
Railway cars employed, 4,572.
Dredges employed, 20.
Cranes, piledrivers, barges, tugs and miscellaneous machines employed, 263.
Total cost, $375,000,000.
First vessel through Gatun Locks, Sept. 26, 1913.
Water let into Culebra Cut, Oct. 1, 1913.
Gamboa dike blown up, Oct. 10, 1913.
First vessel through Miraflores Locks, Oct. 14, 1913.

Culebra Cut.

Length, 9 miles.
Width at bottom, 300 feet.
Width at top, ¼ to ½ mile.
Deepest excavation, 495 feet.
Average depth of excavation, 120 feet.
Excavated by French, 20,419,720 cubic yards.
American excavation, estimate, 89,794,493 cu. yds.
Added excavation, account of slides, about 22,000,000 cubic yards.
Day's record for one steam shovel, 4,823 cubic yards.
Largest slide, Culebra, 10,000,000 cubic yards.

Gatun Dam.

Length, 8,000 feet.
Width of base, 2,100 feet.
Width at water level, 400 feet.
Width at top, 100 feet.
Height, 115 feet.
Volume of rock and clay, 22,100,000 cubic yards.

Gatun Lake.

Area, 164 square miles.
Height of surface above sea level, 85 feet.
Water capacity, 183,000,000,000 cubic yards.
Area of watershed, 1,320 square miles.
Minimum depth, rainy season, 47 feet.
Minimum depth, dry season, 39 feet.

Gatun Locks.

Length over all, 3,500 feet.
Width over all, 350 feet.
Volume of concrete construction, 2,043,730 cu. yds.
Width of side walls at base, 52 feet.
Width of side walls at top, 8 feet.
Width of centre walls, 60 feet.
Height of walls, 81 feet.
Dimensions of lock chambers, 1,000 x 110 feet.
Depth of water in lower lock chambers, 40 feet.
Depth of water in upper lock chambers, 41½ feet.
Length of lock gate leaves, 65 feet.
Height of lock gate leaves, 47.4 to 82 feet.
Weight of largest gate, 1,483,700 pounds.

Editorial

DEPRESSION AND DUTY.

Governments and municipalities, corporations and individuals should strive at this time to remember the natural virtues—justice, prudence, temperance and fortitude. The theological virtues—faith, hope and charity—are equally important, but they will follow without effort if the natural virtues are exercised.

Justice demands that all men who are willing to work be given work.

Prudence requires careful avoidance of extravagance and equally careful avoidance of panic that might be caused by unnecessary retrenchments and false economies.

Temperance embodies rational self-control. It means the suppression of any tendency to thoughtless actions. It implies the calmness and patience that were never more essential in Canada's history than at present.

Fortitude calls for resolution and constancy. It is demanded when the evils of trade depression and war are encountered, if they are to be overcome and transformed into opportunities.

No men in Canada are able to recognize these broad principles of philosophy better than the technically trained and those whose experience has been in handling large numbers of men. Therefore the engineers and contractors have a duty that they should remember. They should be leaders in thought, leaders in action. No man in a community to-day should be bigger than its city engineer, for instance. And the engineers and contractors must direct public thought and action into sane, philosophical, business-like channels.

RAILROADS FAVOR GOOD HIGHWAYS.

Public roads are an indispensible part of the transportation system of Canada, supplementing the railroads and waterways. That this is well recognized by the railroad officials is evidenced by recent remarks made by President Harrison of the Southern Railway Company.

Speaking of the relation of the country highway to the railroad, Mr. Harrison said: "Whatever may be the final destination of the farm products, their first movement must be over the country road, and if the farmer is to receive the largest measure of benefit from good roads, the policy should be adopted of improving those highways which radiate from market towns and shipping stations and over which the farmers must haul their produce. The profit which will be earned by the farmer may depend largely upon the condition of the road from his farm to a shipping station. With good roads he can not only haul heavier loads in shorter time but, except as to perishable commodities, he can market his produce when prices are most favorable and can do his hauling when it is most convenient, and even when the ground is too wet for work in the fields.

"The manifold advantages of an improved highway in reducing the cost of drayage, facilitating social intercourse, promoting school and church attendance, expediting rural mail delivery, increasing the value of farm lands, and promoting agricultural development back from the railroads, are so great that they need but to be enumerated to present a convincing argument in favor of road improvement.

"Several years ago the Southern Railway Company, in conjunction with the United States Agricultural Department and State and local authorities, operated over its lines a good roads train, carrying machinery and lecturers, and building at central points object-lesson roads. This was accelerated in 1911 by the operation of another good roads train in co-operation with the United States Department of Agriculture and the American Highway Association."

CO-OPERATING WITH COMPETITORS.

Trade is rapidly becoming civilized. Corporations, like individuals, have learned that it pays to apply the Golden Rule. No better example of the benefits of proper association of interests could be found than the eleventh annual meeting of the National Paving Brick Manufacturers' Association at Buffalo last week.

The aim of brick manufacturers formerly was to advance as much as possible by individual effort, each plant jealously keeping its competitors in the dark regarding any improvements in machinery or methods, each engineer regarding his knowledge and his experiments as strictly private property.

The result was brick that was not dependable, not uniform, not standard. Co-operation, better ideas, and more liberal business policies have changed all this. Largely through the work of the association mentioned, paving brick has been brought up to a high standard and is being manufactured more efficiently and more economically.

Over two hundred and fifty brick manufacturers and guests, among whom were many municipal and highway engineers and road contractors, attended the meeting. The business sessions were completed on the first day, and the following two days, Thursday and Friday, were devoted to inspections of brick highways in and near Buffalo. There was a free exchange of ideas, both in regard to manufacturing processes and methods of construction. The relative advantages of bituminous and cement grout fillers; the necessity or otherwise of transverse expansion joints, and the best fillers to use if such joints are left in the pavement; the advisability or inadvisability of longitudinal expansion joints at each side of the road, and the advantages therefor of bituminous fillers and of creosoted wood strips—these and scores of other problems of great interest to highway engineers were discussed informally, and many of them practically settled by concensus of opinion.

The discussions were extremely valuable, because over half the men present were brick highway experts—men like "Bill" Perkins, formerly resident engineer at Buffalo for the New York State Highway Commission; Frank Dunn, of Dunn-Wire-Cut-Lug-Brick fame; Will P. Blair, the association's energetic secretary; and scores of other men who, like those mentioned, have built hundreds of miles of brick roads.

The meeting was by no means entirely one of self-admiration, however. It was realized that there is serious work still to be done by the association to perfect brick and methods of brick construction. For instance, the announcement of the State Highway Commissioner of Illinois that he is building 82 miles of road—12 miles of brick and 70 miles of concrete—reminded the members of the association that there are engineers who believe that in building a brick road with cement filler, the bricks should be laid a mile apart.

LETTER TO THE EDITOR.

British Manufacturers Showing Courage.

Sir,—We want to ask you to help us and other British engineering manufacturers by putting a note in your journal explaining that we are keeping our shops going notwithstanding the war, and want all the orders we can get from Canada.

Everybody here appreciates the enormous help we are getting from Canada in men and food, but we want your readers to help to keep our industries alive, so that we can keep the wolf from the door. If Canadians will make a determined effort to send their orders to the old country instead of to the States, it will be invaluable.

Bear in mind that about 25 per cent. of our men are with the forces now and that the rest of us must earn their keep, for the firm has undertaken to pay half salaries and wages to all men who have joined, as well as to keep their places open, and most other firms are doing the same.

We cannot expect to give quite such good deliveries just now as usual, but that is exactly where we look to Canadian buyers to stretch a point.

Sydney (Australia) has set a good example. The corporation had placed an order for a 5,000 kw. turbo-generator with a German firm at a "dumped" price. They have cancelled that order and have placed the work with us at our price and, what is more, they are going to pay us instalments as work proceeds in our shops.

WILLANS & ROBINSON, Limited.

Rugby, England, August 31, 1914.

PERSONAL.

R. H. SPERLING has been appointed assistant to the chairman of the B.E. Electric Railway company, in London, Eng.

WM. C. ROWSE, B.Sc., M.E., has been appointed to the chair in the University of Manitoba, as professor of mechanical engineering. Professor Rowse is a graduate of Purdue University.

H. P. MAYBURY, M. Inst. C.E., in November of last year appointed to the office of Chief Engineer of the British Road Board, has been elected to the newly created position of Manager and Engineer.

M. A. WOODS, recently assistant chief engineer of the G.T.P. Railway system, has been appointed to the position of chief engineer, left vacant by the resignation of Mr. B. B. Kelliher. Mr. Woods' headquarters will be at Winnipeg, Man.

M. C. FLINT, formerly assistant engineer in the construction department of the C.P.R. company, and engaged on branch line and double-track construction, has been appointed resident engineer of district No. 4 of the Alberta division of the C.P.R. system. Mr. Flint's headquarters are at Edmonton, Alta.

JAMES A. MacGREGOR has been recently promoted by the C.P.R. company to a position as district superintendent on the company's Alberta division. Mr. MacGregor's headquarters are Edmonton, Alta. His last office with the railway staff was that of a relieving superintendent on various divisions of the system.

THOMAS ADAMS, the noted town planning expert of the British local government board, is being brought to Canada by the Conservation Commission to act in an advisory capacity. Mr. Adams will arrive in Canada early in October. He will collect information for the commission relative to various Canadian municipalities, and his services will be available to any of them.

L. E. ALLEN, Belleville, Ont., county engineer for Hastings; A. W. ELLSON FAWKES, Calgary, Alta., waterworks engineer; E. C. A. HANSON, Saskatoon, Sask., city electrical engineer; G. H. HATFIELD, St. John, N.B., road engineer; W. MURDOCH, St. John, N.B., city engineer; and G. D. WEAVER, Melfort, Sask., are Canadian names appearing on the most recently published list of members of the Institution of Municipal Engineers (Great Britain).

GEORGE W. COBURN, recently appointed a resident engineer for the C.P.R. company, and located at Brandon, Man., began service with the company in 1896, and since that year, has received the following promotions: from 1896-1900, roadsman and draughtsman at Farnham, Que.; 1901-1907, draughtsman and assistant district engineer, Souris and Brandon, Man., and Moose Jaw, Sask.; 1907-1914, district engineer and resident engineer, Souris and Brandon, Man.

OBITUARY.

The death occurred on August 30th in New York of William De Hertburn Washington, a prominent member of the American Road Builders' Association. He was a member of the American Society of Civil Engineers, and, at one time, was United States consul at London, Ont. Twenty years ago, he went to New York and became President of the Hydraulic Construction Co. Mr. Washington took a prominent part in the Third International Road Congress, held at London, Eng., in June of last year, and also in December at the Philadelphia convention of the American Road Builders' Association.

Word reached the Department of Railways and Canals, Ottawa, on September 10th, of the death caused by drowning on August 25th of James Wilson, an engineer in the employ of the government for the past two years on the Hudson Bay railway construction.

COMING MEETINGS.

ROYAL ARCHITECTURAL INSTITUTE OF CANADA.—Seventh Annual Meeting to be held at Quebec, September 21st and 22nd, 1914. Hon. Secretary, Alcide Chausse, 5 Beaver Hall Square, Montreal.

AMERICAN SOCIETY OF MUNICIPAL IMPROVEMENTS.—Charles Carroll Brown, Secretary, Indianapolis, Ind. Meets at Somerset Hotel, Boston, Mass., October 21st, 22nd and 23rd.

AMERICAN HIGHWAYS ASSOCIATION.—Fourth American Road Congress to be held in Atlanta, Ga., November 9th to 13th, 1914. I. S. Pennybacker, Executive Secretary, and Chas. P. Light, Business Manager, Colorado Building, Washington, D.C.

AMERICAN ROAD BUILDERS' ASSOCIATION.—Eleventh Annual Convention; fifth American Good Roads Congress, and 6th Annual Exhibition of Machinery and Materials. International Amphitheatre, Chicago, Ill., December 14th to 18th, 1914. Secretary, E. L. Powers, 150 Nassau Street, New York, N.Y.