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THE NEW WELLAND SHIP CANAL

THE THIRD AND GREATEST WATERWAY TO JOIN LAKES ERIE AND ONTARIO—ITS GENERAL DESIGN AND COURSE—HISTORICAL NOTES RELATING TO OLD WELLAND CANALS.

AMONG the big engineering undertakings started during the present season there are few that excel in size the structure which is to replace the canal which connects Lakes Erie and Ontario. The Welland Canal now in use has, since its construction, been known as the "new" canal, in distinction from the original waterway which is now historically referred to as the "old" canal, the work under contemplation therefore constituting the third. The original and present canals followed a route from Port Colborne across the Niagara peninsula, practically paralleling each other at the northern end, entering Lake Ontario at Port Dalhousie. The original canal was commenced in 1824, and completed in 1833, the line following very closely certain water courses, to facilitate construction, making the length 27½ miles. This canal contained 27 locks, 24 of which were 150 by 26½ ft., and the other three, 200, 230 and 270 by 45 ft., respectively. The depth of water on the sills was 10¼ ft., with a total lift of 326¾ ft. The initial construction of these locks gave a depth of 8 ft., the increase of 10¼ ft. being a subsequent change. The present canal was commenced in 1872, and completed in 1887. The route at the northerly end was slightly changed to the east near St. Catharines, the canal coming out into Lake Ontario at the same point as the original one. The locks are 26 in number, 270 by 45 ft., with a depth of 14 ft. over the sills.

The route to be followed by the proposed canal, beginning at the southern terminus, i.e., at Port Colborne, is the same as that of the present canal as far as Thorold, from which point it will deviate to connect with a harbor which will be constructed at the Lake Ontario end, some three miles east of the present Port Dalhousie outlet, this diverging portion being about eight miles in length, and passing in almost a straight line through Merritton to the lake. The section of the existing canal between Thorold and Merritton will, therefore, be abandoned, but it is the intention to use the portion of the present canal between that point and Port Dalhousie as an auxiliary.

The present canal is made up of twenty-five locks each 270 ft. by 45 ft. with some 14 ft. of water over the sills. The total length of this canal is 26¾ miles.

The new canal will have seven locks each of 42½ ft. lift, 80 ft. width, and sufficiently long to accommodate a vessel of the length of 800 ft. It will be 25 miles from lake to lake and will cope with a difference of level of 325½ ft. between them. The minimum depth of water over the miter sills of each lock will be 30 ft. The bottom of the canal will be 200 ft. in width, and although the depth will ultimately reach 30 ft., the present excavation will stop when 25 ft. is reached, further deepening being looked after when occasion arises by dredging out the reaches to the required depth, according to design The lock walls will be 82 ft. high above the top of the gate sills, and including the necessary foundation work required below this level two of the locks will have walls 100 ft. high. The lock gates will be of the single-leaf type, swinging on a hinge at one side of the lock and resting in a notch cut in the opposite wall, a single leaf thus spanning the whole width of the lock chamber. The gate at the foot of each lock will be 83 ft. high and 88 ft. long, and will weigh about 1,100 tons. The valves and culverts in the walls will be of large dimensions and will permit of the lock being filled in less than eight minutes. This will mean that the time of passage through the canal will be reduced much below that required at present. The time of lockage of the present canal is from 10 to 20 minutes per lock. In the new canal there will be 18 fewer locks, and the saving in locking time will be in the neighborhood of three hours.

Port Colborne Development.

In Fig. 1 will be noted a proposed new breakwater off Port Colborne, to prevent a disturbance of harbor water, with which the present conditions have to contend, as the present breakwater is insufficient in effect owing to size and location. The new breakwater will consist of an immense rubble mound of stone from the excavation north of Port Colborne, and will terminate in a timber-and-concrete headblock located some 2,000 ft. farther out in the lake than the present breakwater.

The outer harbor at Port Colborne has now a 22-ft. depth of water at ordinary stages of the lake, which is as much as is available at most of the other lake ports and in the channels connecting the lakes at the present time. The deepening of this portion of the harbor may be left for a few years until the connecting channels in the lakes allow deeper navigation. The inner harbor will be excavated to the new depth proposed, and the old locks and regulating weir now in the centre of the village will be entirely removed.

From Port Colborne to Humberstone the rock cut will be deepened and widened on the west side, and just below Humberstone a new cut will be made across the point now forming Ramey's Bend, materially straightening the canal. The excavation from this cut will be principally rock, and will form suitable material for the break-

The present aqueduct at Welland will be dredged out, as well as the bank between the canal and the river. The river will be utilized between Welland and Port Robinson

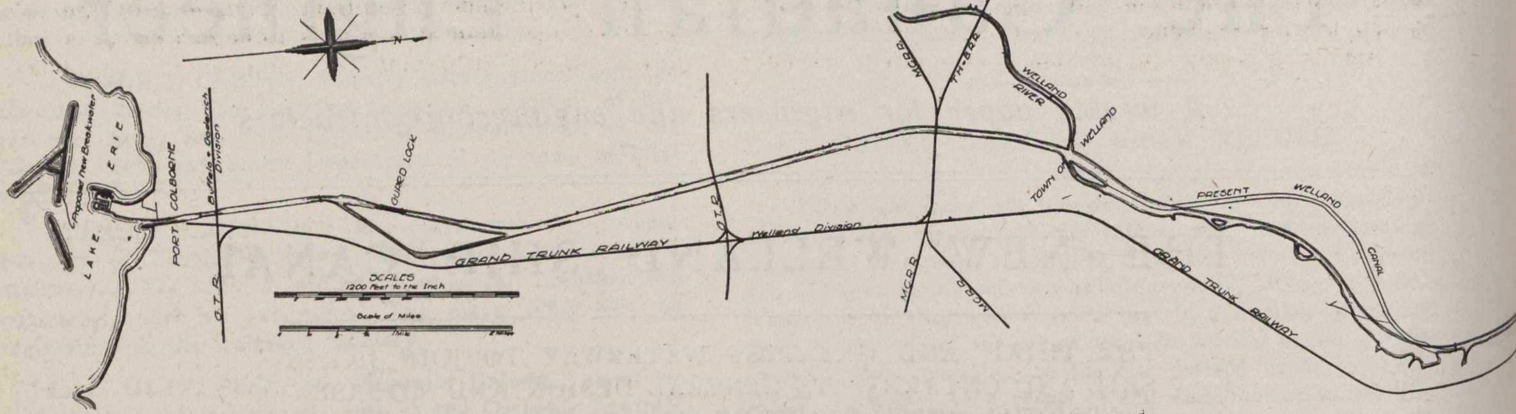


Fig. 1.—Course of New Welland Ship Canal, 25 Miles in Length.

water in Port Colborne harbor.

A guard lock will be built in the rock cutting a short distance below Humberstone, and when this new cutting is ready for navigation a regulating weir will be built across the abandoned portion of the present canal, which will be used as a by-pass to furnish water to the canal. This lock and regulating weir will control the elevation of the summit level of the canal, which it is proposed to keep at the level of extreme low water in Lake Erie, viz., 568 ft. above sea level.

The Welland River Crossing.

From Ramey's Bend to Welland the present canal will be enlarged by an extended excavation along the western bank. It crosses the Welland River by an aqueduct at Welland, as indicated in Fig. 1, but in the new structure it is proposed to raise the level of the river to that of the summit level of the canal by means of a dam

instead of the present canal, being somewhat more direct and entailing considerably less excavation. At Port Robinson a cut will be made through the present bank between the canal and the river, enabling vessels to re-enter the canal prism. Between Port Robinson and Allanburg what is known as the "Deep Cut," which has a maximum depth of 66 ft., will be enlarged by cutting a slice off the western bank.

In raising the Welland River above Welland some 1,600 acres of low land adjoining the river will be flooded. This land is not of any great value, as it is flooded every spring by the river overflow, limiting its agricultural value to pasturage only. The township of Wainfleet, adjoining the Welland River on the south side, consists principally of low-lying ground which drains into the Welland River, and to prevent damage to this land on account of the raising of the river it will be necessary to open up most of the ditches from the point of their pre-

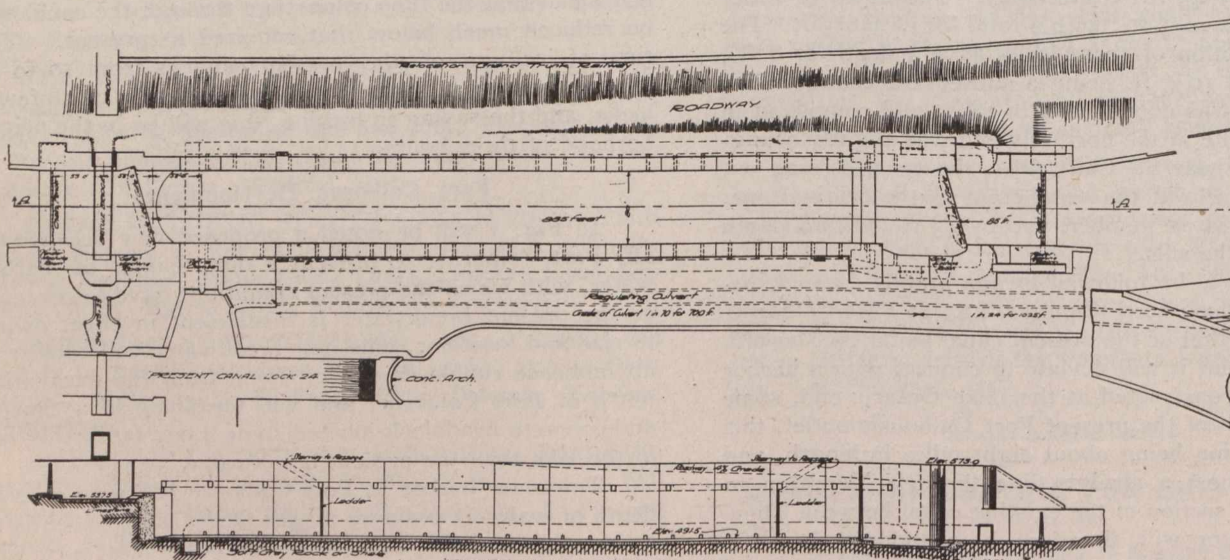


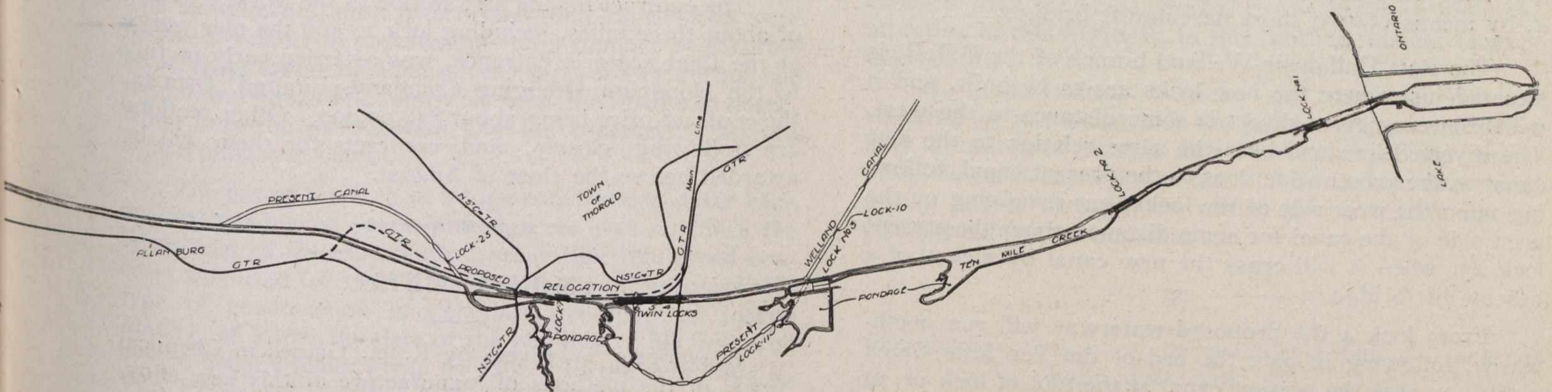
Fig. 2.—General Arrangement of Lock No. 7, Welland Ship Canal.

across the river at Port Robinson. This dam will be provided with a large overflow and regulating weir, which will control the elevation of the summit level, allowing any surplus water to overflow into the old Welland River and pass out into the Niagara River at Chippawa, as at present. A sufficient quantity of water will be allowed to run constantly to keep the river clean.

sent entrance to the river, to the intended high-water mark.

Turning the Welland River into the canal will pollute the waters which are at present used by the towns of Welland, Thorold and Merritton and the city of St. Catharines for domestic purposes. This may necessitate the construction of extensive filtering plants, although this scheme is not looked upon with favor by those interested. An alternative scheme to lay a pipe line from Lake Erie to the reservoirs of the different municipalities, through which clean water would be continuously pumped, is under consideration and appears to be the most feasible scheme available.

Allanburg is now the junction of the present and old



It is Equipped With Seven Lift Locks, Each 800 Feet in Length, With 46½ Feet Lift.

Welland Canals, and the water required for the latter, which is quite considerable, on account of the numerous power developments along it, is taken into the canal through a weir at this point.

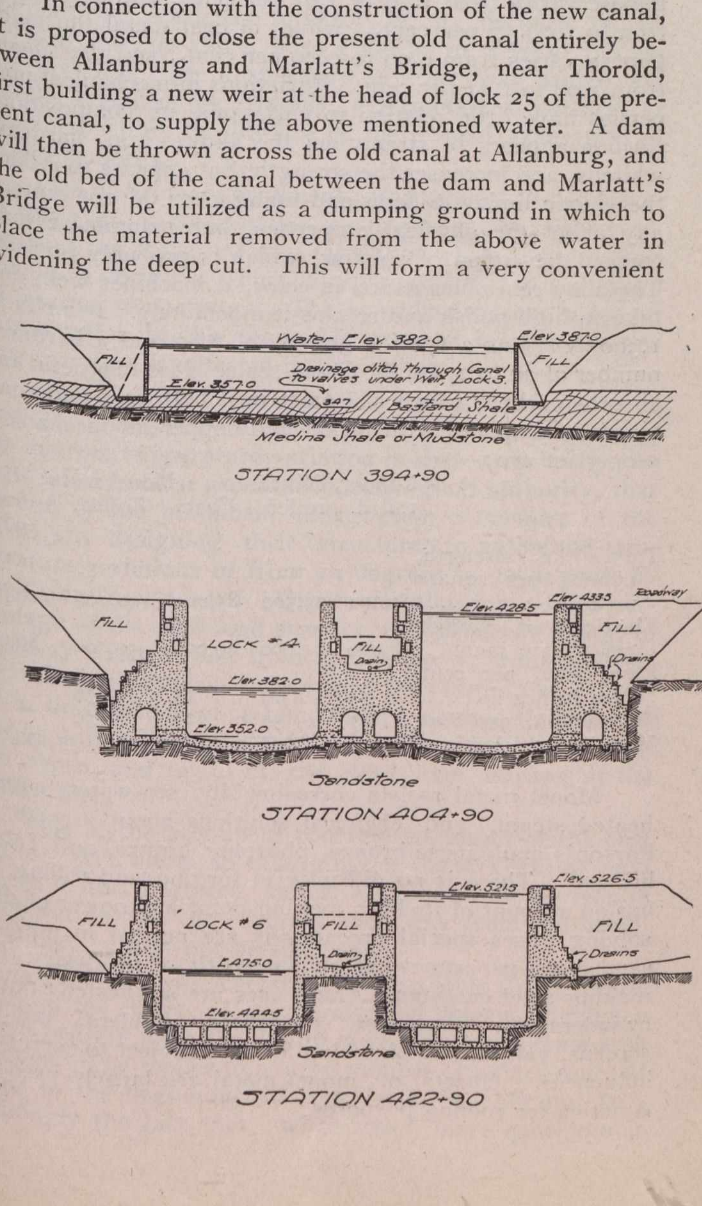
In connection with the construction of the new canal, it is proposed to close the present old canal entirely between Allanburg and Marlatt's Bridge, near Thorold, first building a new weir at the head of lock 25 of the present canal, to supply the above mentioned water. A dam will then be thrown across the old canal at Allanburg, and the old bed of the canal between the dam and Marlatt's Bridge will be utilized as a dumping ground in which to place the material removed from the above water in widening the deep cut. This will form a very convenient dumping ground, and the old canal will become more self-contained, as at present the entrance works are situated at an inconvenient distance from the remainder of the canal.

If it is desired to continue navigation on the old canal, entrance may be had to it through lock 25 of the present canal (a little south of the town of Thorold) when the new canal is completed, by making a short cut through the bank separating the two waterways.

New Lock No. 7.

Between this point and Thorold will be located a pair of twin guard locks, just at the southerly limits of the town, and a short distance north of them will be located lock 7, the head of this lock being directly opposite the head of lock 24 on the present canal. Fig. 2 shows its general design. The portion of the present canal between locks 25 and 24, together with a pond of about 27 acres, formed by flooding the upper valley of the Ten Mile Creek, will be utilized as a regulating basin from which water to fill lock 7 will be drawn. This method of drawing water from a side pond, instead of directly from the canal above, avoids the formation of objectionable currents and surges in the canal and locks, and is the method adopted for filling all of the locks.

Below lock 7 will be a short reach of canal, with an adjacent pondage or regulating basin having a surface



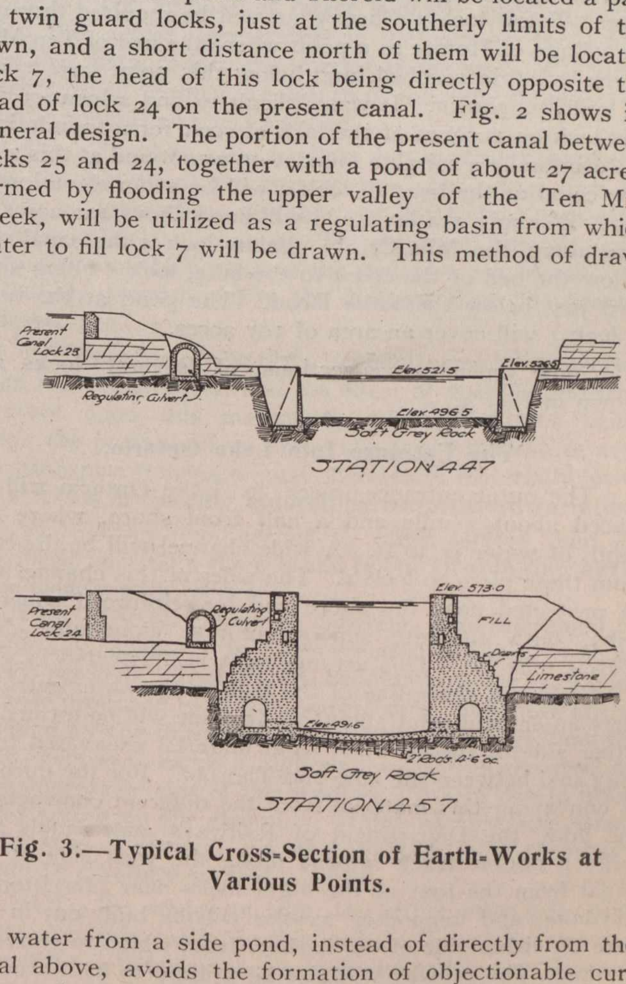


Fig. 3.—Typical Cross-Section of Earth-Works at Various Points.

area of about 84 acres (see Fig. 1), and immediately below will be located twin locks 6, 5 and 4 in flight. These three locks will overcome a descent of 139½ ft. One flight will be used for down bound vessels and the adjoining flight for up bound, a double flight being required to save long delays in the passage of vessels through the canal.

The main line of the G.T.R. between St. Catharines and Niagara Falls will cross over the foot of twin locks 4, by means of two short bascule lift bridges.

The Port Dalhousie-Welland branch of the G.T.R. is situated just where the new locks are to be built, and it will be necessary to divert it some distance to the west. The diverted line will bear the same relation to the new canal as the present line does to the present canal, following upon the west side of the locks, but remaining on the west side of the canal for some distance above the present lock 25, when it will cross the new canal by means of a bascule lift bridge.

From lock 4 the proposed waterway will run northwards, following in part the bed of the Ten Mile Creek until it crosses the present canal at the foot of lock 11, at an elevation of 382 ft. above sea level. This is the level of the present canal at that point, and small vessels may, if desired, use the Port Dalhousie entrance, as at present, as far as lock 11. (See Fig. 1.)

Lock 3 will be located immediately north of the present canal, and at its head on the east side will be situated an equalizing basin or pondage with an area of 150 acres. Below lock 3 a heavy cut will be required through the village of Homer to a point where the bed of Ten Mile Creek is again reached, and below this point lock 2 will be built, as shown. It was difficult to find a location for this lock on account of the lack of rock for a foundation, but eventually a suitable foundation was found at this site. The canal at the head of lock 2 will be at an elevation of 335½ ft. above sea level and will flood about 200 acres of land along Ten Mile Creek. Below lock 2 the canal will follow the bed of the creek to the lake, lock 1 being situated just below the Lake Road. The pond at the head of lock 1 will cover an area of 107 acres.

Some typical cross-sections of several locks are shown in Fig. 3.

The Entrance Into Lake Ontario.

The outer entrance piers in Lake Ontario will be placed about a mile and a half from shore, where the depth of water is 30 ft. A wide channel will be dredged from these piers to lock 1. The sides of this channel will be protected near the shore end by reinforced concrete cribs, with concrete superstructures, alongside which vessels may lie. This arrangement is illustrated in Fig. 1. From the shore line of the lake to the outer entrance piers an embankment about 500 ft. wide will be formed on either side of the channel, from material excavated from the canal between the lake and Thorold. For the purpose of conveying this material from the different contracts to the lake, the Department of Railways and Canals will build a double track railway along the west side of the canal from the foot of the flight locks near Merritton to the lake, and temporary trestles will be built out in the lake on either side of the harbor, from which to start the dumps. The railway will also be utilized to haul crushed stone from the site of the flight locks to locks 1, 2 and 3, for making concrete. The contractor for the rock excavation from the site of the flight locks will, under his contract, be obliged to crush a sufficient quantity of the good rock taken from his excavation to supply all the crushed stone required for making all the concrete for the different locks and structures.

The construction of the new canal is under the direction of the Department of Railways and Canals. It will cost approximately fifty million dollars. The engineer in charge of the entire construction is Mr. J. L. Weller, M. Can. Soc. C.E.

The contract for the first section of the work, a length of about three miles, including lock 1, and the pier works at the Lake Ontario entrance, was awarded early in July to the Dominion Dredging Company, Limited, Ottawa, the contract price being about $3,500,000. Other sections are following closely, and contracts for them will be awarded before the close of August.

MONEL METAL.

A synopsis is given by R. H. Gaines in Chemical News, of the methods of manufacture of this new alloy, the reduction of which has now been so perfected as to give a product of very uniform composition, different analyses of different varieties only varying betyeen the limits: Nickel, 67.55 to 69.54 per cent.; copper, 26.25 to 27.53 per cent.; carbon, 0.17 to 0.44 per cent., and iron, 2.07 to 3.33 per cent. In forged or rolled metal the manganese varies from 1.26 to 1.82 per cent., and the silicon from 0 to 0.37 per cent., while in cast metal the manganese varies from 0.09 to 0.49 per cent., and the silicon from 1.08 to 1.41 per cent. Variations in mechanical properties have little relation to variations in composition. Carbon appears to strengthen the alloy; iron hardens, whitens, and increases the strength, but appears to reduce the elastic limit. Silicon and manganese do not materially affect the physical properties, but manganese may correct the action of sulphur, which is probably present. The alloy resembles nickel in color, it machines well, and takes a high polish and retains it indefinitely. It melts at 1,360° C.; has a specific gravity of 8.87 and a hardness number of 20 to 27 on the Shore scale. It is magnetic and absorbs carbon, which can exist in it both free and combined. Compared with nickel and copper its mechanical properties are:—

	Copper, rolled.	Nikel, rolled.	Monel metal. Cast.	Monel metal. Rolled.	Monel metal. Annealed.
Tensile strength, pounds per square inch	34,000	75,500	85,000	100,000	110,000
Elastic limit, pounds per square inch ...	18,000	21,000	40,000	50,000	80,000
Elongation, per cent. in 2 inches	52	43.9	25	30	25
Contraction, per cent.	57	57	25	50	50
Modulus of elasticity.	....	.....	22,000,000 to 23,000,000		

Monel metal resists corrosion by sea-water, superheated steam, and chemical solutions about as well as Parson's manganese bronze, phosphor bronze, and Tobin bronze. There is no difficulty in forging and rolling it, but on account of its high melting-point and power of dissolving gases special precautions are needed in making sand castings, somewhat similar to those necessary for making steel castings. These uses are suggested: pump cylinders for sea-water, propellers, rudders, mining screens, valves, and plumbing fixtures subject to corrosive influences. Sheets of monel metal are largely used in America for roofing purposes.

THE DETERMINATION OF INTERNAL TEMPERATURE RANGE IN CONCRETE ARCH BRIDGES

BULLETIN No. 30 of the Iowa State College, engineering experiment station, contains a treatment of this subject by Messrs. C. S. Nichols, assistant director of the station, and C. B. McCullough, assistant engineer, Iowa Highway Commission. The experimental data from which their conclusions were deduced, relates to observations on a number of concrete structures, taken by themselves and others during the past four years. The report contains the results in detail, while only the introduction and the conclusions arrived at are published herein.

The use of the arch in bridge construction dates back over two thousand years, although it was not until the latter part of the nineteenth century that reinforced concrete was used for such structures. The invention of this type of construction is generally conceded to Joseph Monier, of Paris, the date of the building of his first arch bridge usually being given as 1867. In the year 1894 F. Von Emperger introduced the "Melan" system of reinforced concrete arch construction into this country, and built the first reinforced concrete bridges of considerable span. Edwin Thatcher was also a pioneer in this work, building bridges as early as this date.

While it is thus seen that the use of reinforced concrete in arch bridges is of comparatively recent date, yet the growing sentiment in favor of it, because of its peculiar esthetic value, is so great as to warrant the belief that this type of structure as a highway bridge will not be replaced by staticly determinate structures, at least not within the lifetime of the present generation of bridge builders.

One of the main arguments used against the adoption of this type of construction is our present limited knowledge of the actual internal forces for which the structure should be designed. Particularly is this true of the range of internal temperature. The immediate necessity for an investigation of this temperature range in arch bridges was therefore manifested in two ways:

(1) There is at present a very great diversity of practice among our western bridge companies in allowing for internal temperature variation in their arch bridge design. It has been ascertained, from good authority, that several of the prominent construction companies of the West are designing their structures to withstand temperature variations of from 15 degrees to 30 degrees F. each way from normal. Others are allowing a fixed percentage of the dead load stresses, this generally being, in highway construction, from 10% to 20 %, while analysis according to the elastic theory for the range to be expected in this latitude gave temperature stresses (in the case of one arch) as high as 206% of the dead load stresses at the crown, and 123% of the dead load stresses at the spring time.

One of the prominent concrete engineers of the east wrote: "We allow, in this latitude, (New York) or anywhere in the northern states, a range of 40 degrees in temperature, or 20 degrees each way from the mean. This may or may not be sufficient."

(2) There are scarcely any available experimental data on the range of temperature in concrete having a direct bearing upon the design of reinforced concrete arch bridges. The replies to a large number of letters sent out prior to the beginning of the tests, brought out very strikingly the fact that, while there were considerable available data and many theories concerning the expansion and contraction in concrete, yet very little was known concerning the actual internal variation of temperature in concrete structures.

Conclusions.—From the results, the writers draw the following conclusions:

I. The yearly range in temperature in a reinforced concrete arch structure, typical of the highway arch construction in this State, is, in this latitude, not far from 80 degrees F.

II. The relation between the depth of concrete covering at any point and the yearly temperature range may be obtained from a curve plotted from the results obtained on bridges under test. The curve that most nearly passes through the centre of the results can be represented by the equation

$$y = 90 - \frac{53}{100} x$$

wherein y = the yearly temperature range in degrees Fahrenheit, and
x = the distance from the nearest exposed surface in inches.

However, the effect of the different factors, such as the presence of a water surface, direction of prevailing winds, etc., so modify the results that the writers prefer to state their conclusions as in I., giving an average value for points throughout a structure of this type.

III. The amount of direct sunlight modifies somewhat the actual temperature in the concrete for a considerable distance into the interior of the mass, although, on account of the meagre nature of the data gathered, no definite conclusion can be stated.

IV. The data seem to show that in structures of this type the minimum temperatures are attained in time intervals anywhere from less than one day to four days after the atmospheric minimum. This interval depends upon the position of the portion of the structure considered, and is roughly proportional to the distance from the nearest exposed face.

V. Because of the high temperature in the concrete when it attains its set, and the effect of atmospheric temperature upon this maximum, other conditions being equal, the pouring of an arch ring at a temperature near the atmospheric mean annual operates to materially lower the stresses in the ring induced by temperature variation.

VI. When uninfluenced by other factors than atmospheric variation, the rise and fall of an arch ring agree quite closely with theory.

VII. The shrinkage of concrete, if unrestrained by reinforcing, amounts, in 100 days after placing, to about 4/1000 per cent. This induces bending stresses analogous to those produced by a temperature drop, but these are so modified by the initial stresses due to shrinkage that the chief effect is to cause a high compression in the steel on the compression side of the bending. When, due to other forces acting on the structure, a high compressive stress in the steel is encountered, the effect of this shrinkage should be carefully studied.

VIII. To render an arch ring structurally safe, provision should be made, in this latitude (Iowa) for stresses induced by a temperature variation of at least 40 degrees F. each way from an assumed temperature of no stress. Particular circumstances may demand that a greater variation be used for drop in temperature to prevent the appearance of cracks. This will always remain largely a matter of judgment with the designing engineer.

A MAINTENANCE-OF-WAY DEPARTMENT RAILROAD TESTING PLANT.*

By B. B. Milner.

SOME time ago the writer was asked to assist a maintenance-of-way engineer in the investigation of a problem which involved a study of track design and the stresses imposed upon its various members. Consultation with several men of recognized experience and authority, as well as an investigation of all experimental work performed in this connection, and a search through the literature of the subject, revealed the fact that dependable data upon which to base definite conclusions upon various points raised were lacking.

The same revelation was made by Mr. O. E. Selby, bridge engineer of the Cleveland, Cincinnati, Chicago & St. Louis Railway, in a paper, entitled "A Study of the Stresses Existing in Track Superstructure and Rational Design Based Thereon," which was published in Bulletin No. 80, American Railway Engineering and Maintenance-of-Way Association, October, 1906. This paper elaborated upon the statement that "railroad track has grown in strength as heavier loads have made increased strength necessary, but such growth has been entirely along empirical lines, and not one single detail of track superstructure bears marks of engineering design."

Mr. Selby, after careful consideration of such factors as rail loading and stress therein, tie bending, bearing of tie upon the ballast, depth of ballast and its bearing upon the subgrade, developed the track design shown in Fig. 1. The sizes on the drawing are for 60,000-pound axle loads. The principal sizes for 50,000-pound loads, using various weights of rail, are given in Table I.

Table I.

Rail	80 pounds	90 pounds	100 pounds	Rail with Sec. Mod. 20
Axle load	50,000	50,000	50,000	60,000
Size of ties	7"x 8"x 8½'	7"x 8"x 8½'	7"x 8"x 8½'	7"x 9"x 8½'
Spacing of ties	16½"	18"	20"	20"
No. of ties 33-foot rail	24	22	20	20
Depth of ballast	14" stone	16" stone	18" stone	12" stone 12" gravel
Width of roadbed	21'	21'	22'	24'

From this table it is at once seen that the number of ties per 33-foot rail, as well as the depth of ballast, is much greater than that found in standard track to-day, and, since the figures in the table are the result of a careful consideration of the strength of the materials involved,

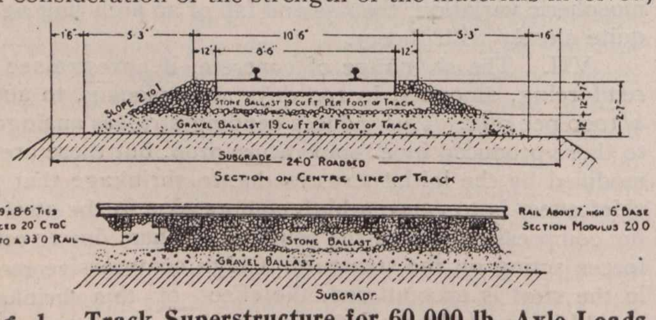


Fig. 1.—Track Superstructure for 60,000 lb. Axle Loads.

it is not surprising that the question of strengthening our present standard tracks is becoming such a live one, especially with our heavier trunk lines.

*Abstract from paper as presented to the Franklin Institute and published in the Journal of the Institute for August, 1913.

The depth of ballast, computed by Mr. Selby, was obtained from formulæ for the thickness of ballast necessary to produce equal distribution of axle loads on the surface of subgrade beneath the ballast, for which Mr. Thomas H. Johnson, consulting engineer of the Pennsylvania Lines West of Pittsburgh, was responsible. Mr. Johnson deduced these formulæ after studying a report, made by Railroad Director Schubert, of Berlin, in 1899, of observations extending over a period of over three years, on the action of ties actually in track. This report was translated and published by Mr. W. C. Cushing, chief engineer, maintenance-of-way, Pennsylvania Lines, and appeared in Bulletin No. 76 of the American Engineering and Maintenance-of-way Association, June, 1906.

In Mr. Johnson's formulæ the two following premises are made:—

1. "That the width of distribution of the load is equal, for stone ballast, to the width of the tie plus the depth of the ballast, and, for gravel ballast, to the width of the tie plus half the depth of the ballast."

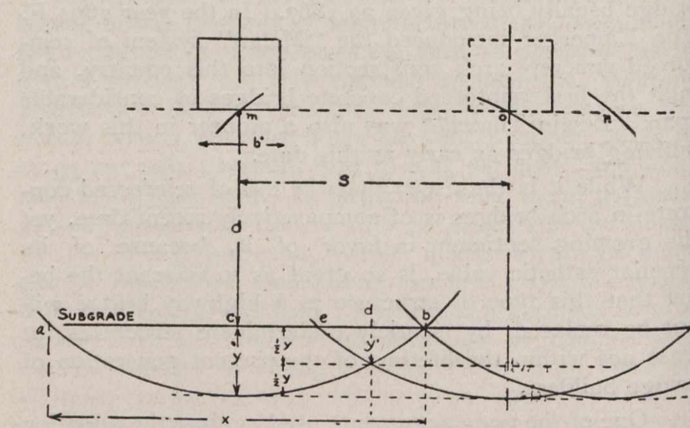


Fig. 2.—Width of Load Distribution Graphically Expressed.

From Fig. 2 this premise may algebraically be expressed thus:—

For gravel, $x = b' + \frac{1}{2} d'$ . . . . (1)

For stone, $x = b' + d'$ . . . . (2)

where x is the width of ballast pyramid carrying the load.

2. "That the intensities of pressure within that width are proportional to ordinates to an arc of a circle whose radius and chord are equal to the width of distribution of the load."

The deduction of the formulæ is as follows:—

If the circular arcs be considered as approximate parabolas, the intensities of pressure may be assumed to be proportional to the ordinates of the curves. The area of the parabolic segment $= \frac{2}{3}xy$, hence the mean ordinate $= \frac{2}{3}y$, or the mean pressure $= \frac{2}{3}$ the maximum.

The pressure at b is 0, hence, to obtain an approximately uniform distribution of pressure over the surface of the subgrade, the tie-spacing S must be such that the curves overlap and have a common ordinate y' equal to $\frac{1}{2}y$. This will obtain when $db = \frac{1}{4}\,cb$; $eb = \frac{1}{4}\,ab$ or $mo = \frac{3}{4}\,mn$.

Hence, the tie-spacing $S = \frac{3}{4}x$.

Therefore, from (1), for gravel, $S = \frac{3}{4}(b' + \frac{1}{2}d')$ and from (2), for stone $S = \frac{3}{4}(b' + d')$ from which the required ballast depths d' are obtained by transposition:

For gravel, $d' = 8/3\,(S - \frac{3}{4}b')$

For stone, $d' = 4/3\,(S - \frac{3}{4}b')$

both of which give values of d' much greater than exist in practice.

The question whether, in order to strengthen our tracks for the increased loads imposed, the number of ties or the depth of ballast, or both, should be increased, is one upon which opinions vary widely among those concerned. The following is here presented in this connection.

Fig. 3 shows a diagrammatic section of the ordinary standard main line track, in which 7-inch by 8-inch ties are spaced 22 inches apart, or 18 per 33-foot rail, and laid upon 6 inches of ballast. The slopes of the ballast pyramids, transmitting the load from bottom of tie to subgrade, are determined in accordance with Mr. Johnson's premises for the distribution of pressure at the bottom of stone ballast. It will be noted that the width of the base of the ballast pyramid under each tie is 14 inches and the width of the strip of subgrade unloaded and lying between adjacent ties is 8 inches. As shown by Director Schubert's experiments, and in line with the experience of all trackmen, the subgrade line, originally straight, will be disturbed, as AB in Fig. 4, the amount of disturbance being proportional to the weakness of the subgrade and to the ratio of the area of subgrade between the loaded ballast pyramids and that of subgrade beneath the ballast through which the load is transmitted.

By changing the tie-spacing to 20 inches, or 20 per 33-foot rail, as shown dotted in Fig. 3, the ratio of loaded to unloaded width of subgrade at the 6-inch depth of ballast becomes 14/6 instead of 14/8, an increase of 33⅓ per cent. The subgrade is more nearly confined, a condition which, of course, materially increases its carrying capacity. Maintaining the 22-inch tie-spacing and in-

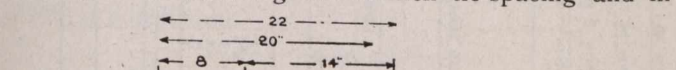


Fig. 3.—Section of Standard Main Line Track.

creasing the depth of ballast from 6 inches to 8 inches changes the ratio of loaded to unloaded width of subgrade from 14/8 to 16/6, an increase of approximately 52½ per cent.; while the combination of the decreased tie-spacing and increased ballast depth increases the original ratio from 14/8 to 16/4, or approximately 128.6 per cent.

In the diagrammatic plan of track shown in Fig. 5 it is assumed that the rail load is spread by the tie 18 inches on each side of the rail centres, or, for an 8-inch tie, over an area of 36 times 8 = 288 square inches of the lower face of the tie under each rail, and over somewhat less than the 36 + 6 times 8 + 6, or 588 square inches of the subgrade. The corresponding area of unloaded subgrade is 42 times 8, or 336 square inches, and the ratio of loaded to unloaded subgrade areas is 588/336. Decreasing the tie-spacing to 20 inches will increase this ratio to 558/252, or 33⅓ per cent., while combining with this decrease, an increase of ballast depth of 2 inches, will increase the ratio to 588/168, or 100 per cent.

Fig. 5 also shows that with 18 ties per 33-foot rail the unloaded area of subgrade between adjacent ties becomes zero when the depth of ballast is increased from 6 inches to 14 inches, while with 20 ties per 33-foot rail it becomes zero with a ballast depth of 12 inches.

According to Johnson's formula for stone ballast, uniform distribution of subgrade loading will be obtained with 21⅓ inches of ballast for 18 ties per 33-foot rail and with 18⅔ inches of ballast for 20 ties per 33-foot rail.

Any consideration of the relief to be expected from decreasing the tie-spacing, increasing the depth of ballast or otherwise, should take into account the cost, and in this connection Table II. is presented.

From this table it is seen that a reduction of tie-spacing from 22 inches to 20 inches increases the ratio of width of loaded to unloaded subgrade by 33⅓ per cent. at a cost of $464, $928, and $1,856 per mile for single, double, and four-track lines respectively, while an increase of ballast depth of two inches (from 6 inches to 8 inches) increases this ratio by 52½ per cent. at the respective cost of $256.90, $507.23, and $1,006.51 per inch additional inches of additional ballast. Whether the first or the ballast, or $513.80, $1,014.46, and $2,013.02 for the two

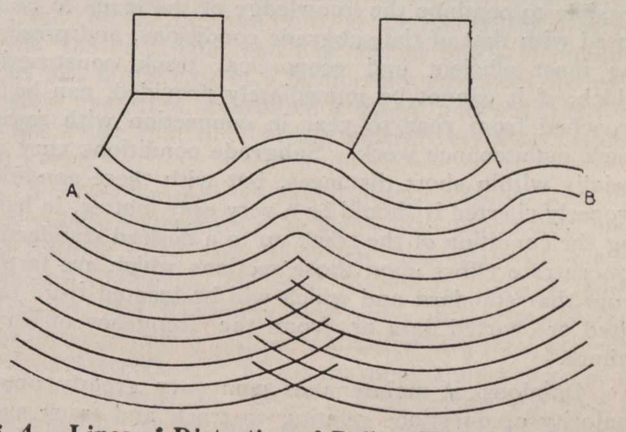


Fig. 4.—Lines of Distortion of Ballast Under Loaded Ties

second or a combination of both is best is therefore somewhat of an open question.

Some individuals and committees have recently recommended a ballast depth greatly in excess of previously existing standards and still greater than the ballast depths actually obtaining even on our densely travelled lines, without qualification dependent upon either the subgrades or the characteristics of the imposed loads.

Table II.

Statement Showing Comparative Cost of Increasing Number of Ties per Rail Length (33 Feet) from 18 to 20 and Cost per Inch Depth of Putting Additional Ballast Under Standard Track of One of Our Larger Lines.

Ties per mile spaced	Single track.	Double track.	Four-track.
18 per rail length	2,880	5,760	11,520
20 per rail length	3,200	6,400	12,800
Additional ties	320	640	1,280

Cost per tie:

1 tie..........$0.90		Volume of 1 tie, 3.3 cu. ft.
2 plates....... .28		Cost of ballast, 75cts. per cu. yd.
8 spikes ...... .11		Cost of laying ballast, 40 cents per cubic yard.
$1.29		Total cost of ballast in track, $1.15 per cubic yard.

Credit account ballast displaced			$0.14
Net cost per tie			$1.15
Labor placing tie in track			.30
Cost per tie in track			$1.45
Cost of additional ties in track	$464*	$928*	$1,856*
Full width of ballast base	164½"	324½"	644½"
Cubic yards per square inch section per mile equals 1.35			
Cubic yards ballast required	223.39	441.07	875.23
Cost of required ballast in track, at $1.15	$256.90	$507.23	$1,006.51

To the writer it appears that the design of railroad track must be approached in the same way as the design of any mechanical parts. As a foundation it should be treated like any other foundation, and a study of its requirements, purposes, construction, and maintenance should be made. It must be designed to carry loads of certain individual magnitude, density and speed, etc., and must do this upon subgrade of given conditions. With a sufficient amount of the right kind of data, it should be possible to combine the knowledge of the loads to be imposed with that of the subgrade conditions, and prescribe the most efficient and economical track construction, which, if it cannot be immediately provided, can be approached from year to year in connection with regular track maintenance work. Subgrade conditions vary materially within short distances, but with these conditions properly charted it should be a very easy matter, in bringing the condition of the track up to a desired standard, to concentrate effort upon those sections which are farthest from that standard and which will be located from tabulated or charted data or from the frequency of service failure.

Opinions of maintenance men vary greatly upon a majority of questions relating to track and track superstructure design, and practically none of these opinions is supported by such conclusive test or experience as will make early agreement or reconciliation possible. In the maintenance-of-way department dependable data are not being collected as in the motive power department, in which department much money has been, and is continually being, profitably spent to settle questions pertaining to the design and operation of both locomotives and cars, many of which questions are of less importance than some of those relating to track construction.

The writers upon maintenance-of-way subjects will have served a useful purpose if they succeed in focusing attention upon the comparative deficiency of experimental work in this field, for as soon as this deficiency is realized then work will be inaugurated which will, in a few years, elevate the science of track construction to its proper level. The condition of our tracks is now lagging behind requirements, and we cannot consistently hope for a better state of affairs unless some move along scientific lines is soon inaugurated. In the meantime, locomotive and car designers must "mark time," so far as increased loads and speeds are concerned.

*Does not include labor cost of respacing present ties to make the insertion of additional ties possible, nor the cost of distributing the ties and ballast to point where placed in track.

While some investigations should necessarily be made under regular service, many can, nevertheless, be (and a few have been) carried out upon an experimental track of significant proportions. An experimental track was constructed a comparatively short time ago by the Prussian State Railroads at Oranienburg, Germany, and is being used to determine experimentally the best construction. Dr. H. K. Hatt, of Purdue University, who visited this installation a few years ago, reported:

"It consists of an oval track two miles in circumference over which runs a train consisting of electric locomotive and cars. At about every fifty feet two vertical rail ends were sunk in the ground on each side of the track and clips riveted onto them to serve as a reference

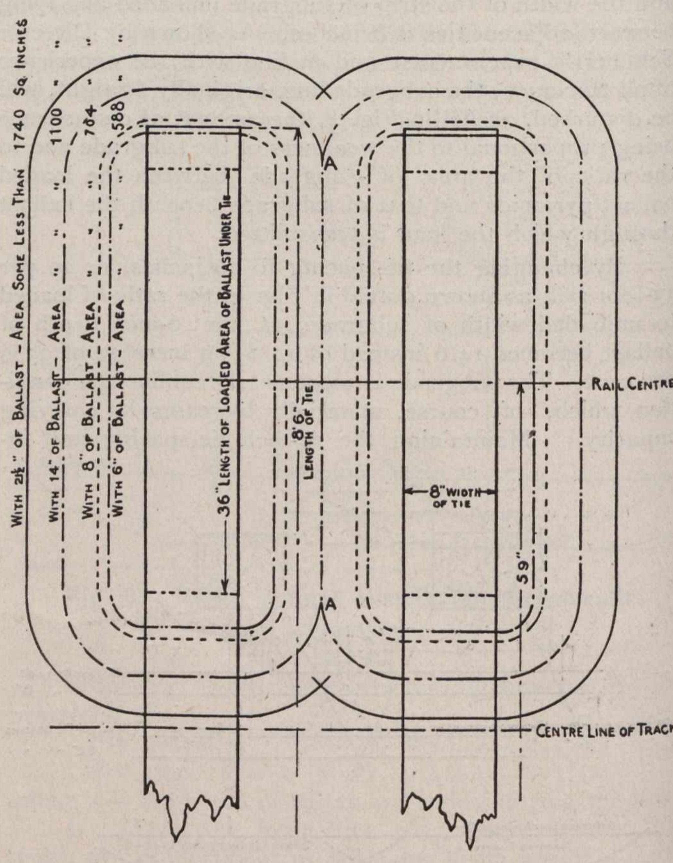


Fig. 5.—Sketch of Track Plan.

7" x 8" x 8' 6" ties spaced; 22" or 18" per 33' rail. Showing outlines and areas of loaded sections of subgrade with 6", 8", 14" and 21½" of ballast under the ties.

line, for measurement of track deformation. The service is considered as severe in one year as eight years on the main line. The cost of the roadbed and equipment is stated to have been $40,000.

"About a year before my visit the first track had become worn out and a new track consisting mainly of steel and beech ties had been set under service.

"Some of the elements which were being experimented with at Oranienburg may be listed as follows: Prussian standard steel ties with side ribs have been down one year in different forms of ballast. The evidences of derailment of cars were visible, but the steel ties were not sprung. The modern double steel tie was used at the joints. Various forms of anchorage of rails to ties were under experimentation. These anchors are found more necessary in steel ties than in the case of wood. The various forms of anchors seemed all to be effective, although the track as a whole had not been down long enough to give final results.

"The scarf joint appeared to be unsuccessful, inasmuch as the inner edge of the scarf sheared off at the edge, due to the wave motion in the rail. The records of traffic passing over this experimental track show that in 369 working days the number of kilometre tons was about six million. In the months of January, February, March and April, in 1909, there were nearly seven million kilometre tons. The locomotive weighed 59½ tons and would pull a train of from 240 to 375 tons. The speed of the train was 60 kilometres per hour."

The loads imposed upon this experimental track are insignificant in comparison with those which, for the results obtained to be of service, would have to be placed upon such a track in this country.

The writer would suggest that an experimental track be designed and constructed in the form of a figure 8, so that one loop could be operated while the other was being prepared for test. On this track, properly enclosed by fencing, etc., a motor, hauling any desired combination of cars, loaded with any desired axle loads, could be controlled from the office of an engineer of maintenance-of-way tests located in the centre of either of the loops, and in this way any combination of rails, fastenings, ties, joints, ballast, frogs, switches, signals, and safety devices could be tested. The value of an ability, by this method, to rapidly test rails of different steels, under more scientific conditions, must be apparent when the amount of work done in settling rail questions during the last ten years is considered.

CANADIAN PETROLEUM.

The petroleum industry in Canada has been marked during the past four years by a rapidly decreasing output, the production in 1911 being only a little more than one-third that of 1907, according to statistics contained in the annual report on mineral production of Canada, by John McLeish, B.A. The total production of crude petroleum in 1911 was 291,092 barrels of 35 imperial gallons each, valued at $357,073 or an average of $1.22½ per barrel, as compared with a production of 315,895 barrels valued at $388,550, or an average of $1.23 per barrel in 1910, and 420,755 barrels valued at $559,604, or an average of $1.33 per barrel in 1909. With the exception of 86,139 gallon in 1911, 51,975 gallons in 1910, and 3,328 gallons in 1909, produced in New Brunswick, the output was entirely from the Ontario oil fields.

The above statistics of production are based on the claims made for the bounty paid by the Dominion Government, which was first provided for in 1904 by an Act passed by the Dominion Government authorizing the payment of a bounty of 1½ cents per gallon on crude petroleum produced from wells in Canada. The bounty was continued during 1910 under the "Petroleum Bounty Act, 1909," which provides for the payment of bounty on crude petroleum produced from oil-shales mined in Canada, as well as on oil from wells in Canada. Payments are made on claims submitted by the producers of crude oil to the Minister of Trade and Commerce. These claims have to be substantiated as to quantity by the certificate of the receiving stations, tanking companies refiners or other purchasers, as well as by the supervising officers of the Department of Trade and Commerce.

The bounty paid on the crude petroleum produced gives, therefore, as accurate a basis as is available for a reliable statement of the annual production.

A FEW COMPARATIVE COSTS IN ROAD AND PAVEMENT WORK.

By Fred. L. Macpherson, A. M. Inst. C.E.
Municipal Engineer, Burnaby, B.C.

Although the cost of roads and pavements naturally varies greatly in each city, or district, depending on local conditions, labor and materials, a comparison of the average prices obtaining in the vicinity of Burnaby, B.C., may be interesting.

Macadam Road.—Initial cost, 80 cents per sq. yd.; annual cost of oiling, .02 cent per sq. yd.; average annual cost of maintenance during the first three years of life, 11 cents per sq. yd.; cost applied over three years, $1.19 per sq. yd.; cost of scarifying and resurfacing usually necessary at the end of three years, 32 cents per sq. yd.

This means that in the fourth year of its existence an ordinary macadam road under average conditions has cost $1.50 per sq. yd., and even then it is far from being a permanent structure, being still susceptible to patching, scraping, oiling and resurfacing continually, thus increasing the cost of the road until, at the end of the fifth year or so, the macadam road has cost as much, if not more than, the initial cost of a bituminous pavement for which a usual five years' guarantee is given, and which is likely to last twice that time with comparatively little maintenance cost.

Further figures may be conclusive evidence of the extravagant construction of ordinary macadam roads. During the last three years there has been spent on our two principal highways, Douglas Road, 6¼ miles in length, $110,000, and Kingsway, 4¼ miles in length, $60,000. The latter is now being reconstructed at a total estimated cost of $400,000, for which the maintenance (except the cost of cleaning) of the actual pavement will cost the municipality practically nothing during the next ten years. Had this road not been paved, the repairing, resurfacing, oiling, etc., during that time would have cost $50,000—a very conservative estimate. Unless for purely residential streets, where it best fulfils its purpose, the ordinary macadam road should be a thing of the past. The present policy of borrowing money for 40 years for building roads with an average life of three years, and maintaining such roads out of revenue expenditure, seems to be radically wrong. An ordinary macadam road, far from being an asset, is nothing short of a burden, under such conditions.

The average cost of other pavements dealt with are:

Concrete pavements	$2.00 per sq. yd.
Hassam pavements, thickness, 6 ins.	2.25 per sq. yd.
Granitoid pavements, thickness, 7 ins.	2.65 per sq. yd.
Brick on city streets on 6-in. concrete	3.75 per sq. yd.
Brick on rural roads on 4-in. concrete	2.75 per sq. yd.
Dolarway pavement	2.20 per sq. yd.
Bituminous pavements—	
On concrete base	2.60 per sq. yd.
On old macadam base	1.25 per sq. yd.
On new macadam base	1.80 per sq. yd.

Based on an average cost of $25,000 per mile, 100 miles of bituminous pavements built on existing macadam or gravel roads in Burnaby would cost $2,500,000. While the initial cost of such work seems alarming, the investment should be a sound one if spread over a period of about five years, inasmuch as the work would be permanent, the average annual cost of maintenance, say, $50,000, at the end of that period would be practically eliminated and the needs of every district amply served.

Although on first consideration such an undertaking would appear to be stupendous and financially impossible for any ordinary municipality, the issues at stake are so serious, and the necessities so imperative as not only to deserve mature consideration but also to demand prompt action. Permanent initial construction is the proper policy for any municipal or government authorities to adopt. Municipal bodies are active in the permanent road policy, but unfortunately the action is not concerted enough and the results are therefore disappointing and ineffective.

The proper administration of highways is a prominent feature of road economics which has not had the earnest consideration locally which its great importance deserves. In view of the increasingly urgent demands for the opening up of new roads for settlers, and the improvement of existing residential roads, a municipality should not altogether be saddled with either the construction or the maintenance of through highways. Such roads and the roads to which they link up should be under a highway commission, composed of engineers and representatives of some more permanent public bodies, such as is in vogue in England and in portions of Eastern Canada and United States. The adoption of such an administration would assuredly make for uniformity and continuity of construction, and result in a more even and equitable assessment of expenditures involved.

The provincial governments appear to be alive to the situation and have adopted progressive permanent measures in their road construction policy. Financial assistance is now given chiefly for permanent work, and, locally, Burnaby and South Vancouver have reason to be greatful for the material assistance received in the permanent paving of Kingsway.

There are many other interesting and important features associated with the question of semi-permanent and permanent roads and pavements, but the above remarks may serve to prove that progress towards the better highway and the better street, in which we are all so intimately interested, must be along lines that are utilatarian, logical and aesthetic, and further, that the wish for a more permanent highway will be visionary until the need for it is created and its claims urged upon the powers that be.

* The total value of the sales of sewer pipe in 1911 statistics as contained in the annual report on Mineral Production of Canada by John McLeish, B.A., was $812,716, as compared with a value of $774,110 in 1910, and a value of $645,722 in 1909. Nearly 50 per cent. of the production in 1911 was made in Ontario. Following is a list of firms that reported production of sewer pipe in 1911: Standard Drain Pipe Company, St. Johns, Que., and New Glasgow, N.S.; Ontario Sewer Pipe Company, Toronto, Ont.; Dominion Sewer Pipe Company, Toronto, Ont.; Hamilton and Toronto Sewer Pipe Company, Limited, Hamilton, Ont.; Clayburn Company, Limited, B.C.; British Columbia Pottery Company, Victoria, B.C. The imports of drain pipe and sewer pipe during the calendar year 1911 were valued at $382,929, of which $338,644 worth was imported from the United States, $44,278 from Great Britain, and $7 from other countries. The production of drain tile as reported was not as large in 1911 as in 1910 or 1909. The total sales in 1911 were valued at $339,812, as against $370,008 in 1910, and $408,440 in 1909.

FOREST DEVELOPMENT AND PRESERVATION.*

By R. H. Campbell, Esq., Director of Forestry, Ottawa.

IN considering the forest and its development, it is necessary to have clearly fixed in our minds the fact that the forest is a living thing, and that it grows and develops according to natural laws. In order to understand the forest, therefore, it is necessary to study the habits of life of the tree. The four great requirements for tree life are water, light, air and soil. The tree carries on its functions of life and growth very much in the same way as animals do. One of the great differences, however, is that the tree is in a fixed position and therefore cannot go to find its food or requirements where they may be. It is therefore necessary for it to adapt itself to the conditions as they exist where it is located. We will look at the materials which the tree requires for its growth, and then consider some of the adaptations that are necessary so that the tree may make use of them.

Water is the most important constituent for the use of the tree. The tree is continually drawing up from the earth through its roots, large quantities of water which pass up from the trunk and are exhaled from its leaves. In order to keep the tree in a healthy condition this flow of water must be kept constant, and the more freely the water passes through the tree the more vigorous will be its growth. We see this very clearly by following different characteristics of the tree growth in different portions of the Dominion. On the coast of British Columbia, in a moist climate, the growth of the tree is very vigorous and we have the immense Douglas firs and cedars that form one of the most magnificent forests in Canada. In the forests of the Selkirk Mountains in the central part of British Columbia where the rainfall is still heavy, there is a vigorous growth of white pine, cedar and fir. In the dry belt in the province of British Columbia, the mountains which receive the most of the precipitation are covered with a dense forest of lodgepole pine and fir, but in the lower portion of the dry district, where the rainfall is scanty, the forest thins out and there is a scattered growth of yellow pine. In the prairie country the Rocky Mountains carry a stand of lodgepole pine, Englemann's spruce and fir, which is more or less of a mountainous character, and therefore not so large in size as the forests of the Selkirk Mountains or the coast of British Columbia. The reason for this is that ascending the mountains or going northward in latitude the soil is cold for a longer period during the year, and this has a considerable effect on the rate of growth of the trees.

The trees are not directly affected by the lowness of the temperature, as some specimens of trees are found growing in the barren lands to the north, away beyond the general line of tree growth. The effect of the cold is, however, to keep the ground frozen for a great portion of the year, and therefore to prevent the tree from obtaining a supply of water with the result that is cannot continue its functions. In the muskegs of the north country the trees are very slow in growth and stunted. This is partly the result of the fact that the ground is kept very cold by the covering of moss, and therefore the growth is slow, but it is also partly due to the fact that there is considerable acid in the water of the muskegs and consequently

*Read at the Seventh Annual Meeting of the Association of Dominion Land Surveyors, Ottawa, March 5th, 1913.

the tree cannot make use of the water as it could if it were pure. The method by which the water passes from the outside of the rootlets to the inside is a physical one dependent on chemical composition. If the water outside is a stronger solution than that inside the root, the water will not pass in and consequently the tree cannot get the use of it.

As you reach the northern latitudes or the alpine elevations of the mountains, the tree growth becomes more stunted until finally it lies almost like a mat on the surface of the earth, as illustrated by the view of the white barked pine at an altitude of 6,000 feet on the Porcupine Hills in Alberta. This flattening out of the growth is partly due to the fact that the water supply is not sufficient, and that it is difficult to get the water pumped up if the tree grows to a greater height than the ordinary level, and partly to the fact that the drying wind increases so largely the transpiration that the supply of water cannot be obtained quickly enough in a soil which is so cold for such a great part of the year.

To counterbalance the great extent of the tree above ground, it is necessary also that the root should extend deep and wide in the earth in order that the tree may not be overthrown by the wind. The growth of the tree in this manner is not, however, the most satisfactory for producing lumber, as the lumber produced will be knotty and will have a very small proportion of clear stuff. In order to produce the best quality of lumber the tree should be grown in a close stand. Where the trees have been sown close together and are reaching upward toward the light, each one tries to get its crown where it can get the most light. As a result the trees grow rapidly upward and the inside branches dry off and gradually drop from the tree, leaving a long, clear trunk. Many of the trees in such a stand will finally become suppressed by the more vigorous ones and will gradually die out completely. In the forests of Europe such trees would be taken out by a thinning process, but in this country the market conditions do not make such thinnings possible, although in a district like the Cypress Hills, of Alberta, where there is no other timber within one hundred miles, there seems to be reason why thinning might be done profitably.

The soil has considerable influence on the growth of trees. We find that on the light, sandy soils the pine will grow readily, particularly the red pine and the jack pine. As a rule, however, trees will grow best in a fairly good soil, and the forest floor covered by leaves and debris from the trees finally forms a bed of humus which provides excellent nourishment for tree growth. The province of Ontario is now taking active steps to replant pine on some of the sandy areas in the western part of the province, which have been entirely denuded and left bare of trees or any other useful growth.

In order to meet the different conditions as explained above, the tree has to arrange various adaptations. Sometimes the tree is in a location where water can be obtained plentifully, and in such case it is arranged that water will pass through the tree rapidly and escape. Such trees usually have broad leaves with thin coverings and numerous openings called stomata from which the moisture may escape. Trees or plants growing in dry situations, on the other hand, have devices for retaining the water as long as possible. One of the clearest examples of this, is the cactus growing in the deserts of the western United States, and even in the dry districts in the southern part of the prairies. The cactus has thick leaves containing a great many water vessels, and these leaves are covered with a close covering through which water cannot pass very easily. As a consequence all the water which the plant draws up from the earth is retained in the plant with almost no loss.

While the tree draws up water from the earth, bearing a few mineral salts, its chief sustenance is the carbon drawn from the air through the leaves. The fact that the tree is largely composed of carbon can be seen in any burnt district. The carbon is supplied largely from carbon dioxide in the air. In order to carry on its process of obtaining the carbon from the air, which process is carried on by assimilation in the leaves, it is necessary that light should play upon the leaves as fully as possible. Consequently the leaves are spread out so as to catch as much light as they possibly can. This is illustrated by the broad, flat leaf of the maple, arranged upon the stem so that the broad, upper portion of the leaf should always be exposed as much as possible to the sun. In the case of the coniferous trees the leaves are small, but this deficiency is made up by their leaves being very numerous.

The stem is so arranged as to provide for the water supply from the root passing up readily, and it is a very interesting process to investigate. It is not yet clearly understood how the force is supplied which will drive up the water required for one of the giant trees on the Pacific Coast two or three hundred feet through vessels of microscopic size. Various theories in regard to the matter have been put forward, and it was thought at one time that the matter was settled by an explanation that the pulsations of the living cells of the trees resulted in a pumping process which gradually forced the water upward. This theory is not, however, accepted as final at the present time.

Each year of growth adds a distinct ring to the woody tissue of the tree. In the spring the cells of wood are large with thin walls, and late in the summer are small with thick walls, and this makes the distinction between the growths of different years. The growth of a year may be larger or smaller according to the season. The favorable season will make the ring of growth larger, and a dry season will make it considerably narrower. Trees which are suppressed will also make slower growth than those which are open to light and air.

The bark gives protection to the growing parts of the trees, and as the tree increases in size the outer bark dies and is split open by the pressure from the inside, resulting in the corrugated bark which is so characteristic of many trees. The rough bark is characteristic of the white pine, elm and many other species. The bark in other cases, like the spruce, strips off in flakes, and that of the white birch exfoliates in thin sheets.

There is added to the natural conditions which have to be considered in the growth of the trees, an artificial condition caused by fire. Fires have occurred in all our wooded districts, and the evidence of their destructive power is clear to any person who has travelled through the timbered districts. Fires make the condition more difficult inasmuch as they make it harder to carry out the protection of a forest, and that they have made the conditions for reproducing the forest more difficult than they would be naturally. Where a tract is well covered with forest trees the cutting can be done so as to favor natural production, but where a forest fire sweeps through, the conditions are changed without regard to what is to follow, and we have to try afterwards to bring about the conditions that would have been considered and provided for if the removal of the forest had been done in a regular way. The vast stretches of forest that have been destroyed by fire mean a great loss to the capital and industry of the country, and means for combatting them are of prime importance.

The methods taken for handling the forest on better principles are:—

1. The provision of a fire patrol. We have a large number of fire rangers patrolling the forests by foot, by horse, by canoe or boat, during the dangerous season in the summer. These men put up notices warning people of the danger of fire, and instruct every person that they meet as to the necessity of taking care in the use of fire. On the Saskatchewan River, and one or two other places, we have put on a patrol boat. In such cases there are large stretches of water from which the forest can be reached and in this way a very much larger area can be patrolled than can be done in any other way. The work of the fire patrol may be illustrated by the work that is done in the portion of Manitoba north of Lake Winnipeg. This district is in charge of Mr. Jas. T. Blackford, with headquarters at Norway House. The canoe routes for the country to the north and down towards Hudson Bay all start from Norway House, and here he has an opportunity of meeting the parties that are travelling and warning them in regard to the danger of fire when they are starting out. Fire notices are posted at all the portages along the canoe routes, so that the matter is brought immediately to the attention of any parties when they go into camp. Mr. Blackford has also been successful in interesting the Indians in the matter of fire protection. He has talked to them about the matter when they are gathered for treaty payment and has induced them to promise to put out their own fires and have others do the same. Every one of the Indians who promises to do this receives a small fire ranger's badge in the shape of a maple leaf, and they are very proud of these badges and wear them all the time.

On the forest reserves where matters are more fully under control, and the line between agricultural and forest lands is permanently settled, we take further steps for fire fighting by making permanent improvements of various kinds. In a tract like the Cypress Hills, where the prairie comes immediately in contact with the forest, plowed fire guards are made so as to prevent prairie fires sweeping into the timber. Such fire guards have been found of great advantage in holding back fires. Telephone lines have been erected in order to have communication through the forest with the object of getting word about the fires out quickly and getting help in to fight them. In the forests of Europe the telephone is being largely used, and it is also being used very considerably on the forest reserves of the United States. It is one of the most useful adjuncts to the fire preventive work that can be had. The work of fire prevention in the forests is on the same principle as the work of fire prevention in the city; that is, the fire must be located quickly; notice of it must be given without delay; and help must be got to it speedily. This means the opening up of communication of all kinds—trails, roads, telephone lines, etc.

Another matter that has to be dealt with in order to meet the fire difficulty is the disposal of slash resulting from lumber operations. The condition of the forest after lumbering operations is such that the ground is covered with the debris of tops and branches forming fuel for a fierce fire and one which it is almost impossible to fight. The methods proposed for disposing of this debris is to have it piled and have it burned at a season when there is no danger of its spreading. Another method is to have the lower and upper branches of the top lopped so that the top will fall flat on the ground and rot away more quickly. This reduces the time of the danger by a few years, but until the tops rot it leaves a dangerous situation. On the whole the best method will be to pile the tops and burn them, and in Minnesota it has been found that this work can be done for 25 cents per thousand feet board measure. In Europe, all material produced by the forest is of value so that the question of the disposal of slash is not a difficult one there. It will be many years before that condition is reached in Canada.

In order to provide for replacing cut over forest by a new growth there are two main methods; one is to do the cutting so as to leave seed trees or groups of seed trees so that a supply of seed may be furnished for the reproduction of the trees. In Canada the natural reproduction is good, as a rule, and as the method of artificial reproduction is somewhat costly, the former method will be followed generally. However, in some cases the forest has been so depleted on lands that are non-agricultural that it is necessary to make arrangements for reforestation. Such a tract is that in the Spruce Woods Forest Reserve in Manitoba, and we have started a nursery there with the object of growing young trees and replanting the denuded areas from the nursery. The method used is that of establishing seed beds in the nurseries and transplanting the trees into the larger beds before they are finally ready to plant out in the forest at the age of about three years. When the trees have reached that age they are planted out in the place where they are to grow finally. In the western prairie district the trees are usually planted about four feet apart in order to give them as much protection as possible. Planting at this distance will mean that considerable thinning should be done at a later stage, and possibly by the time the trees have reached that stage thinning will be a commercial possibility. In many cases, however, the trees are planted as much as eight feet apart.

The whole object of the work that is being undertaken is to finally produce forests where there is a regular stand of clear timber producing a large quantity of wood regularly every year. The danger of not looking after the tree growth is in evidence on the bare and denuded mountain sides of many places in China where forests once flourished; on the bare, rocky hillsides in the Holy Land where a few small shrubs and grasses are gathered for fuel; and on the bare mountain sides of several districts in France where millions of dollars had to be spent for works to control the flow of torrential mountain streams and to replace the forests which have been destroyed by fire and close grazing.

The American Bar Association and the American Bankers' Association have each authorized special committees to co-operate with the management of the American Road Congress in the holding of sessions at Detroit to deal respectively with road legislation and the financing of road improvement. An effort will be made at the legislative session to bring about the formation through official channels of a national committee to codify State road laws and recommend simple and uniform legislation for each State and the elimination of the great accumulation of conflicting and confusing road laws. The finance section will endeavor to have adopted in all parts of the country simple and effective methods of road accounting and a record of such cost data as may be essential to the proper conduct of road work, and will also deal with the important subject of bond issues.

PREVENTION OF WASTE OF FLOWING OIL AND GAS.

METHODS for the prevention of the tremendous waste that usually accompanies the "coming in" of wells producing large quantities of oil or gas by natural flow fall naturally into two classes, preventive and remedial. The first have to do with keeping under control whatever gas or oil may be encountered in the process of drilling; the second relate to the capping or subduing of wells that have "gone wild." Both are dealt with fully in a bulletin prepared by R. Arnold and V. R. Garfias, and recently issued by the U.S. Bureau of Mines. Of these the preventive methods are likely to be of interest in the Canadian field, and are outlined here with that in view.

A few general conditions affecting flowing gas and oil wells are as follows:—

1. The gas from gas wells and the gas that generally accompanies the oil is not injurious to health, hence workmen may labor near the well without being harmed.

2. Owing to the usual tremendous velocity of the stream of gas, the part immediately over the casing head is like a smooth column, and may be approached with safety.

3. The sand usually expelled under tremendous force with the oil or gas often wears out the casings, but this wearing action generally takes place only throughout the uppermost 30 or 40 feet and opposite or near the place where the gas enters the well, thus leaving the main part of the entire length of the strings practically sound.

4. The great volume and tremendous pressure of the gas and oil make the use of the best fittings a necessity. Some of these, being of special sizes, are made to order.

For convenience, and in accordance with common usage, the flowing oil wells and the gas wells discussed herein are designated respectively as "gushers" and "gassers."

With the present state of our knowledge regarding the situation of the "gusher" and "gasses" strata in the developed fields, there is no excuse for the general lack of precautions taken before the depth is reached at which the flow is expected. Furthermore, if the well is being drilled in a new or prospective field, adequate precautions should be taken so as not to jeopardize in advance the possible profits of such an uncertain and expensive undertaking. The additional cost of the safety devices is insignificant in comparison with the total cost of drilling such wells and with the amount that can usually be saved if their rate of production is regulated. It may be safely stated that practically all of the great waste resulting from the unrestricted flow of gassers and gushers can be prevented easily by known means which generally are within reach.

Controlling the rate of production of the well has a direct bearing on the returns to be derived from the product. There are times when, owing to the market conditions and the transportation and storage facilities, the "bringing in" of the well might be an unwelcome event, as was strikingly shown in the case of the Lake View gusher. Experience also shows that a gasser or gusher will yield a greater total production if allowed to flow only a fraction of its capacity, the famous Huasteca No. 7 well in the Mexican oil fields affording a most remarkable example. This gusher has been flowing under perfect control for about 2½ years, yielding about 23,000 barrels per day under a pressure of about 285 pounds.

One of the most successful apparatus devised with a view to controlling whatever flow of oil or gas is encountered is the blow-out preventer, which has given general satisfaction, particularly in drilling by the rotary method.

Description of Apparatus.—The blow-out preventer is a modified type of a stuffing-box casing head, which in turn is an addition to the four-way **T**. The immediate object of the preventer is to control the flow of oil or gas between two strings of casing, and naturally its use is better adapted to drilling by the rotary method in which the circulating water keeps an open space between the inner casing and the wall of the hole. An idea of its operation may be gained by imagining a four-way **T** attached to the top of the last casing set or landed and the inner casing passing through the **T**, the flow of oil or gas between the casings being deflected to the lateral openings of the **T** by closing the annular space between the inner casing and the upper part of the **T** by means of a stuffing box. In the preventer the stuffing box can be removed or put in place by turning adjuster screws which operate slips provided with hydraulic packing that fit around the inner casing as an adaptable split valve. To the lower opening are bolted interchangeable screw flanges to fit standard casings from 6 to 12½ inches in diameter, and to the opposite opening are attached suitable guide plates to prevent the wearing of the inner parts of the apparatus by the revolving inner casing. These guide plates are made to fit casings from 8 to 2¼ inches in diameter. The lateral openings are threaded to fit pipe 6 inches in diameter. The apparatus is manufactured of cast steel and lined with babbitt metal, being tested to stand a pressure of about 2,000 pounds per square inch.

Mode of Operation.—To insure the successful operation of the preventer, careful attention must be given to all the details, such as welding the bars, fitting the small pins that secure the wrench to the screws, and providing a suitable place outside the derrick for the man who is to operate the wrenches, etc., as soon as the apparatus is put in place.

The preventer, when used, is always placed on the last string of casing set or landed, thus controlling the flow between that string and the drill pipe or casing that is being lowered. The slips should be in a position to be clamped around the drill pipe, but should not be kept too near it, in order to avoid possible damage to the hydraulic packing. If the well is being drilled with a rotary rig, the back-pressure valve inside the drill pipe will effectively prevent any flow upward through this casing, and if the standard rig is used, a heavy gate valve, which can be closed after removing the drilling tools, is placed on top of the inner casing. The only outlet left is the space between the casings, and the flow through this opening can generally be controlled by clamping the split valve of the preventer around the drill pipe and closing the gate valves on the pipes connected to the two lateral openings.

It sometimes becomes necessary to exclude from the drill hole water or gas found in an overlying sand in order to control the well or to recover or test the oil below. This end can be easily accomplished in drilling with a rotary rig by forcing muddy water to the bottom of the well through one of the lateral openings of the preventer, the pressure being regulated by closing the valve connected to the other opening. A back-pressure valve in the drill pipe will prevent the flow of the mixture through this casing. By drilling a short distance and forcing as much mud as possible into the porous sands penetrated, a plastered wall of clay is built throughout the gas-bearing or water-bearing strata. After the entire thickness has been penetrated and the casing landed in an impervious bed, another string may be used to tap the oil sands below. The gas thus excluded can be recovered afterwards by drilling shallower wells to the upper sands.

This method can also be employed in drilling the entire thickness of a gas or oil sand under tremendous pressures, an operation that might otherwise prove troublesome. The mud can easily be removed by releasing the pressure or bailing down the water in the hole, a strainer or perforated casing being placed in the sand to prevent the wall of the well from caving when the pressure is released.

Testing of Supposed Oil or Gas Sands.—It is often advisable before drilling is continued to ascertain whether certain beds carry oil. The work involved is often accompanied by accidents and loss of time, as the material may collapse and pack around the casing, thus preventing its removal or further lowering. Many such accidents can be avoided and the required tests made by using a preventer or a stuffing-box casing head in the following manner.

After the preventer has been attached to the last casing set, a special shoe about 8 feet long is attached to the bottom of the inner casing, and this is lowered to the bottom of the hole and driven down into the formation, making a water-tight joint. The split valve of the preventer is then closed. A pump is connected to one of the lateral outlets, the valve on the other being kept closed to control the pressure in the well. Muddy water is then pumped in, and the pressure between the casings is increased in order that the formation will not cave in and "freeze" the inner casing. While the pressure is maintained the column of fluid in the inner casing is bailed out. This fluid being a representative sample of that which came from the formations passed through should show the presence of oil if any has been encountered. Great care should be exercised in bailing, as the casing may happen to be in such condition that it will not withstand the outside water pressure.

Precautions Against Fires.—Perhaps the greatest individual losses in connection with flowing oil and gas wells have been caused by conflagrations which in most instances could have been avoided by proper precautions. In such conflagrations the gas, owing to its lower flash point, is generally set on fire before the oil. Ignition may be caused by fires under boilers or in forges, by unprotected lamps, burning matches, or spontaneous combustion at the bearings of the calf wheel, bull wheel, headache post, or band wheel in the engine house. It is also claimed that the friction of pebbles striking each other or rubbing against the casing as they are expelled with great force with the gas or oil may cause sparks that will set on fire the more inflammable hydrocarbons. In certain oil fields fires caused by lightning are comparatively numerous.

The prevention of these fires depends on the proper control of the flow of gas and oil, the ventilation of the space immediately above and near the well so that no gas will accumulate, and the keeping of boiler fires, forges, lamps, burning matches, poorly insulated electric wires, etc., at such a distance or in such a position as to eliminate danger.

The most effective means employed for subduing gas or oil fires has been the use of steam discharged in great quantities and under high pressure against the lowest part of the flame. Well cappers have also been used successfully. Nature at times comes to the rescue by temporarily stopping the flow when the well "sands up." On the other hand, the flow may render human ingenuity powerless by destroying the casing and forming a veritable oil volcano, as was the case in Mexico with the famous Dos Bocas well.

UNITED STATES CEMENT PRODUCTION, 1912.

The United States Geological Survey will shortly issue a bulletin, "The Mineral Resources for 1912," which will contain the following data concerning the quantities and values of cement produced in that country during the past three years:—

Class.	1910. Quantity (barrels).	1910. Value.	1911. Quantity (barrels).	1911. Value.	1912. Quantity (barrels).	1912. Value.
Portland	76,549,951	$68,205,800	78,528,637	$66,248,817	82,438,096	$67,016,928
Natural	1,139,239	483,006	926,091	378,533	a 821,231	367,222
Puzzolan	95,951	63,286	93,230	77,786	a 91,864	77,363
Total	77,785,141	$68,752,092	79,547,958	$66,705,136	83,351,191	$67,461,513

a Shipments.

Prices.—According to reports made to the Geological Survey by the manufacturers, the average price by districts of Portland cement per barrel in 1912 in bulk at the mills ranged between 67.4 cents in the Lehigh district and $1.358 on the Pacific coast, as compared with 71.5 cents and $1.406 for the same districts in 1911. The average price for the whole country was 81.3 cents in 1912, as compared with 84.4 cents in 1911, a decrease of 3.1 cents a barrel. The average price in 1912 reached the same level as in 1909, and is the lowest point recorded for Portland cement. The lowest average price reported to the Survey was 62 cents a barrel, received at several plants in Pennsylvania, and much cement was sold as low as 65 cents, not only in the east, but in the middle west. The highest figure reported was $1.65 a barrel, reported by a California plant.

A subject which will be taken up at the contractors' session of the Road Congress, at Detroit, which should prove of special interest to road contractors is the "Protection and Upkeep of Road Equipment." An immense amount of money is lost through failure to protect and properly maintain costly road machinery and equipment. Another subject which will be presented by a trained specialist is the "Organization and Arrangement of Working Forces." It is quite a problem to place the road hands of a big force so as to avoid waste of time and money, and this subject should bring out much useful discussion. A special committee of three of the most prominent engineers and contractors identified with contract work, of which Mr. J. R. Wemlinger, secretary American Society of Engineering Contractors, is chairman, will have charge of the programme of the contractors' session.

FOUNDATION WORK ON C.P.R. BRIDGE AT MUD LAKE, ONTARIO.

THE Canadian Pacific Railway Company's low-grade freight line, now building between Toronto and Perth, Ont., crosses Mud Lake some 150 miles north-west of Toronto, and foundations for a single-track bridge at this place are now nearing completion. The bridge, when constructed, will have a length between the parapet walls of the abutments of 953 ft. 7½ ins., and an extreme height above rock foundations of 165 ft. Fig. 1 is a general diagrammatic sketch showing design. It will be supported upon four piers, two sets of pedestals of four each, for column support, and two abutments. The bridge site is some six miles from the railway station. This signifies that the entire plant as well as supplies has had to be transported over land by wagon over hilly country, and since the main plant, aside from such auxiliary plants as are ordinarily used on such work, consisted of three 90-h.p. Loco boilers weighing some 20 tons each, and two air compressors of practically the same weight, the difficulty attending the installation of plant may be understood.

Geological Condition of Site.—Mud Lake lies in a pocket surrounded on three sides by high hills. It derives its name from semi-liquid ooze lying in a mass almost from the surface of the water to the hard-pan some 86 feet below it at the lowest point, which is where Pier No. 2 is located, presenting a formidable opposition to the sinking of caissons under air on account of its low bearing value both in friction and in compression, while in density it could well be classified as solf, blue clay. Despite all its depth of mud to be contended with, the route across Mud Lake chosen by the railway engineers presented to them fewer difficulties than an alternative location among the granite hills surrounding it; and besides, it enabled them to maintain that distinctive feature of the new route, i.e., its low-grade line.

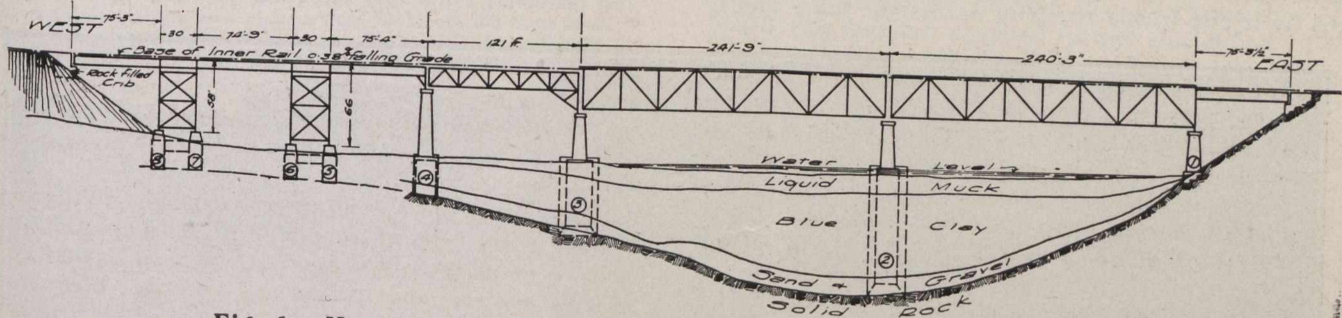


Fig. 1.—New Canadian Pacific Railway Bridge Over Mud Lake, Ont.

Active work was begun late in March of this year and was continued night and day since then until the beginning of August.

Fig. 2.—General View, Showing Piers No. 1, No. 3 and No. 4 Completed.

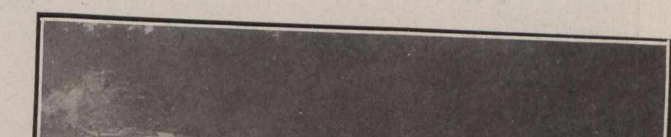


Fig. 3.—Caisson for Pier No. 2 Being Sunk.

Of the four piers, Nos. 2, 3 and 4 were sunk under air pressure: No. 2 to a depth of 103 feet 4 inches, No. 3 to a depth of 59 feet, and No. 4, 40 feet. Except for the last thirteen feet, which consisted of quicksand and boulders running into hardpan, the materials which these piers replaced were liquid muck and solf, blue clay. The foundations for Pier No. 1 and pedestals Nos. 5, 6, 7 and 8 were sunk in open sheet-piling cofferdams.

All foundations and piers are being constructed on concrete in proportions of 1:2:4 and 1:3:5 and rest upon solid rock. Sand for the concrete is procured from a pit some two miles from the bridge site and is delivered by wagon. Stone is quarried and crushed at the bridge site.

The mixing plant consists of a one-yard steam mixer located near the west end of the bridge. The concrete is transported in dump buckets on cars over a track supported by a pile trestle reaching across the lake, and is deck, which was accomplished under water, being by no means an easy task, the remaining 63 feet being sunk under air. In this deck the shafting was embedded and carried to the surface above water level, on the top of which the air lock was placed.

Caisson No. 3 was the same size, only the depth being so much less makes description of its progress unnecessary.

Sinking a caisson to a depth of 103 feet 4 inches, as in the case of Pier No. 2, requiring a maximum air pressure of 47 lbs. to the square inch, is an undertaking that does not permit of methods other than those well tried and proven. Particular care was exercised for the safety

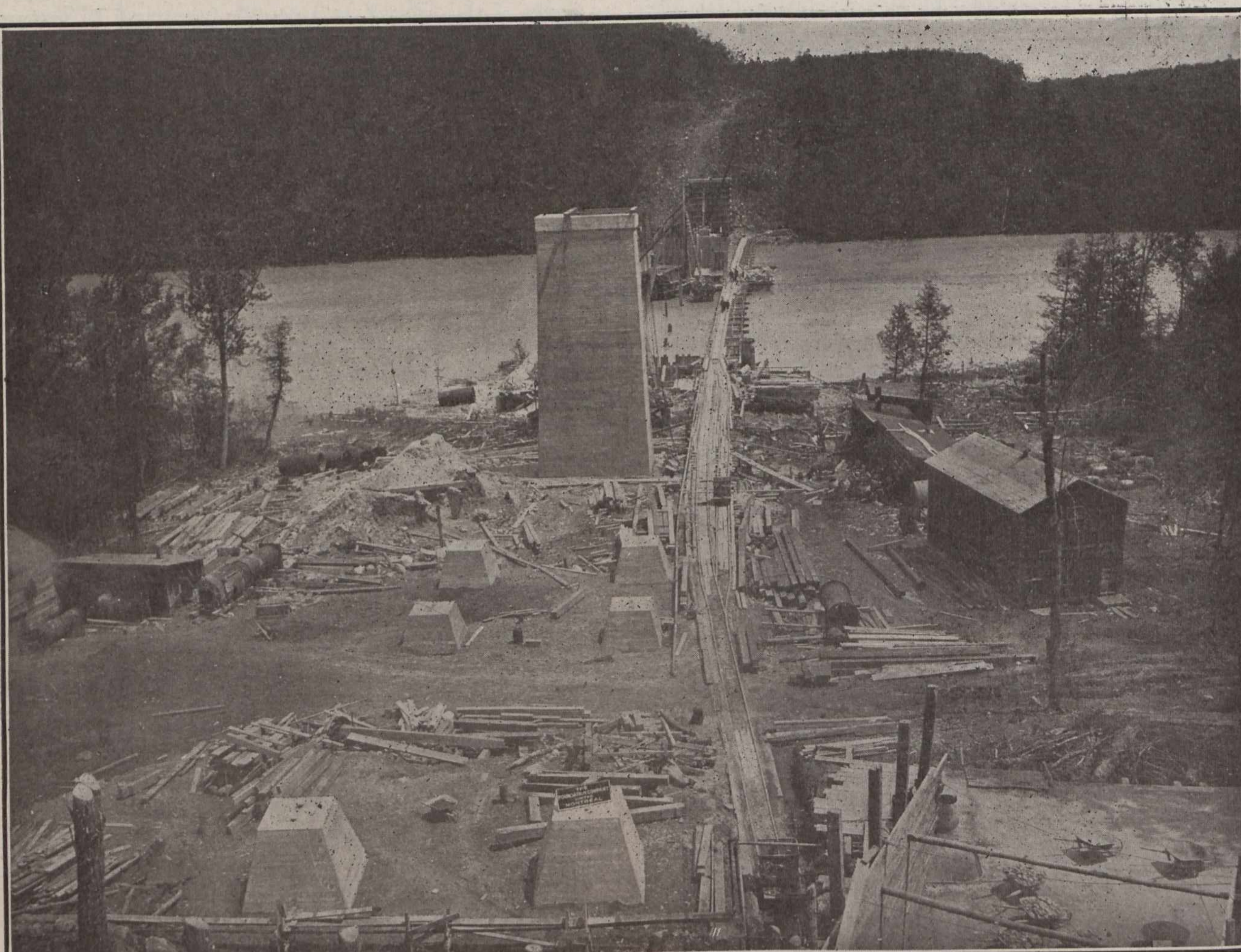


Fig. 4.—Bird's Eye View Looking East, Showing Pedestals for Steel Work, and Pier No. 4 in the Foreground.

handled into the work by means of stiff-leg derricks.

Figs. 2, 3 and 4 show general arrangement of plant, shore conditions, and progress of the work up to about three weeks ago. The massiveness of the concrete piers is well indicated.

The deepest of the two caissons sunk by pneumatic method, viz., Caisson No. 2 was 34 x 20 feet outside dimensions, provided with two dredging wells 8 ft. 8 in. by 9 ft. inside dimensions. Owing to the soft nature of the material and the despatch with which it was necessary to sink these caissons, they were sheeted and braced like an ordinary wood caisson with steel cutting edge, the walls of the working chambers being reinforced with rods. The elimination of forms thus enabled the sinking to be carried on continuously night and day.

After the caisson was dredged to a depth of 40 feet, it was deemed advisable to place the reinforced concrete of labor employed on the work. Only men with previous experience on caisson work were engaged. The time interval to which they were subjected to this excessive air pressure was forty-five minutes, and in spite of all precaution, careful attention and surgical aid, the "sand hogs" were not all the time immune from the "bends" and paralysis. A hospital lock was in commission at Mud Lake, and those afflicted during the sinking of the caissons were speedily treated by the usual air method.

The work at Mud Lake was carried out under the direction of C. W. P. Ramsay, Esq., engineer of construction, from design of P. B. Motley, Esq., engineer of bridges, for the Canadian Pacific Railway. The foundations for Mud Lake bridge were built by The Foundation Company, Limited, of Montreal and Vancouver, Mr. A. I. Campbell, superintendent.

The Canadian Engineer

ESTABLISHED 1893.

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CONTENTS OF THIS ISSUE.

MUNICIPAL ENGINEER AND EMPLOYER.

The uncalled-for subordination which some members of civic boards would like to see their engineers exhibit, is, to the unprejudiced and fair-minded observer, an apt illustration of the fact that engineering and its problems are but minor details, in comparison with other difficulties with which the municipal engineer is confronted. A case was recently cited in the Western press. It relates to the indignant tone of a committee meeting at which the engineer's report upon a certain phase of construction work was being read. A point, with which the engineer had apparently taken for granted that every member of the board was familiar, required further elucidation. The engineer being absent and efforts to locate him failing, an alderman abruptly declared that it was about time to call for the engineer's resignation, and for a general cleaning-up of his department (the latter being a favorite phrase among some aldermen).

Inquiry was rewarded by the information that the engineer had taken suddenly ill and was removed to a health resort. Prior to his departure his deputies were carefully instructed in the matter of their work during his absence, and the department was experiencing no cessation of activities as the result of his personal misfortune. With the promise of his medical adviser to acquaint the Board of Works of his necessary suspension of duties, the engineer had obeyed the strict instructions, thereby occasioning the debate already referred to, in which more than one alderman is quoted to have endorsed the statement, "If a man in his position (the engineer's) goes away without our instructions and without our knowing when he is coming back, it is about time to call for his resignation."

The scene changes. More recent press reports state that during the comparatively short period of twelve months, six working months of 1911 and 1912, this engineer has established a record in development and improvement work in that city which has not been equalled in any other Canadian city, his record including over 32 miles of block pavement, over 56 miles of sewers, about 95 miles of cement sidewalks, some 74 to 80 miles of streets cleared and rough-graded, 30 to 40 miles of rocking, and about 90 miles of water mains, the aggregate cost of this gigantic achievement being in the neighborhood of $8,000,000. In addition to this successful endeavor, through bringing down recommendations for greater yardages of pavements than had hitherto been attempted, and thereby inducing keener competition, he saved the city something like $130,000, as compared with the cost of the pavements which had previously been put down.

The engineer was not fired, thanks to the discrepancies in the point-carrying ability of some individuals, but his discharge was under consideration without an appearance of good grounds.

The case is just another example of the absolute lack on the part of civic councils of the elementary principles of fairness in dealing with municipal officials. Other examples have been previously given in *The Canadian Engineer*. Each example endorses the opinion that city councils have outgrown their usefulness, that the qualifications which bring about the election of many of the members are far removed from those of knowledge and ability which the responsibilities of their positions warrant, and that so long as the present method of election is entertained, these governing bodies will not measure up to a much higher standard than the one that is being referred to here. As already mentioned time and again in this journal, the trouble lies in the city councils and the remedy is there also.

It is against such contingencies as these that some municipal engineers are obliged to carry on a single-handed strife. It is very doubtful whether the existence and powers of a professional association on their behalf, to whose consideration they might refer such occurrences as these, would have much weight, considering the idiosyncrasies of many members of present-day municipal governing bodies.

ENGLISH FOR ENGINEERS.

Much has been written at one time and another about the utility of English as part of the engineering curriculum. *The Canadian Engineer* has always held that there is a need for some training for engineers along this line, in addition to that which the high school course supplies. Perhaps this need could not be better summed up than in the following paragraph, which we take the liberty of quoting from an article by Benjamin P. Kurtz in the California Journal of Technology:

"When facts of iron and steel, of girders and trusses, are turned into a written report, they are presented no longer in their own tangible, objective medium or material, but in the new and subjective medium of words. Now, in the first place, it is to be noted that in this passage from the realm of objective reality to written representation there is ever present the dangerous chance of deflection of facts and even of their actual transformation—in a word, the danger of error. This, merely because of the sudden transition to a new and unaccustomed medium. More narrowly regarded, however, the difficulty arising here is that of accuracy of statement plus the adaptation of technical facts and information to the laws and economy of mental attention. In order to gain and hold the attention of the reader, in order to present facts in such fashion that they may be easily and thoroughly understood, and that the general proposition may be seen to be supported at every point by its details, so that there is the mutual proof of a complete harmony between parts and the whole—in order to accomplish this successful communication with another mind, it at once becomes necessary to marshal the objective facts or material in such fashion that they will find a ready, orderly and emphatic entrance into the mind of the reader. Facts without grouping dissipate the attention; poor grouping, overlapping division, insufficiently marked separation confuse the attention; diffuse, wandering connections weary the attention; neglect to distinguish between division and implication, or between fact and hypothesis, muddle the conception of the reader; tortuous and nebulous sentences befog the conception of the writer; insufficient recognition of the necessity of exemplification and illustration, and ignorance of their difference, leave the reader too much to do or undo; the very lack of knowledge of what constitutes a definition, and of the fundamental methods of expanding a logical definition, lay the entire argument open to objection or render its outlines amorphous. In two words, the necessity of being understood, not the achievement of truth; the necessity of presenting groups of facts in accordance with the habits of trained thought-attention, not the accuracy of turning one fact into one phase;—this is that new labor and skill required of the technician when he expounds his facts and thoughts to other minds in the medium of words."

This very clearly states the newer conditions to be met. We regret that the value of what Mr. Kurtz calls "expository composition" has not been as fully recognized by the engineering faculties of some of our universities as to provide for a course in English of as great a length as can reasonably be expected in their already-crowded curriculums.

CANADIAN PUBLIC HEALTH ASSOCIATION.

The third annual congress of the Canadian Public Health Association will be held in Regina, Sask., on September 18th, 19th and 20, 1913.

The following papers will be presented at the sectional meeting of engineers and architects, of which Mr. L. A. Thornton, B.A., B.Sc., city commissioner of Regina, is chairman:—

(a) "Municipal Loans and Municipal Works," R. O. Wynne-Roberts, M. Inst. C.E., M. Can. Soc. C.E., consulting engineer, Regina.

(b) "Hygiene of School Buildings," J. H. Puntin, L.R., I.B.A., architect to the Regina School Board.

(c) "Results of Water Filtration at Saskatoon," Geo. T. Clark, B.A., A. M. Can. Soc. C.E., city engineer, Saskatoon.

(d) "Sanitary Surveys of Rivers," J. R. Malek, C.E., assistant sanitary engineer, Bureau of Public Health, Regina.

(e) "Sunlight in Cities," Theo. Brockmann, Dipl. Ingr. (Berlin), city engineer's office, Regina.

(f) "The Effect of Water Filtration—Biological and Chemical," H. W. Cowan, C.E., Toronto.

(g) Paper—title to be announced later—J. Antonisen, C.E., superintendent, Brandon Municipal Railway.

(h) "Canadian Conditions Affecting the Design, Construction and Operation of Sewage Disposal Works," R. H. Murray, A. M. Inst. C.E., A. M. Can. Soc. C.E., resident sanitary engineer, Bureau of Public Health, Regina.

In addition to the above, the following papers of particular interest to engineers will be presented at the general sessions:—

"The Smoke Problem," R. N. Blackburn, Wh.Sch., chief inspector of steam boilers for Saskatchewan.

"Standards With Reference to Sewage Treatment," T. Aird Murray, M. Can. Soc. C.E., consulting engineer, Toronto, Ont.

Engineers and public health officials intending to be present at the congress should notify the local secretary, R. H. Murray, C.E., Bureau of Public Health, Regina.

LETTER TO THE EDITOR.

A Question of Stability.

Sir,—The following account of how a large brick arch is standing at the present time, with parts of its abutments entirely cut away, is a very good illustration of the remark once made by a great engineer who, in response to a query as to why a certain structure stood up replied, "By the grace of God and the force of habit," and for this, if for no other reason, the following description should be of interest to all engineers and contractors.

Extensive alterations are being made to a church situated on the west side of the city of New York. These alterations, in so far as they have been carried out up to the present time, consist in entirely wrecking one wing of the church, in order to make way for a new addition. The wrecked wing was separated from the main body of the church by a 28-inch brick bearing-wall, which has been left in position, except as hereinafter described. Communication between the two parts of the edifice through this wall was by means of a large opening about 18 feet wide, spanned by a four-rowlock brick arch. This arch not only carries the 28-inch wall extending for about 15 feet above the crown of the arch, but also carries a small portion of the roof of the main part of the church.

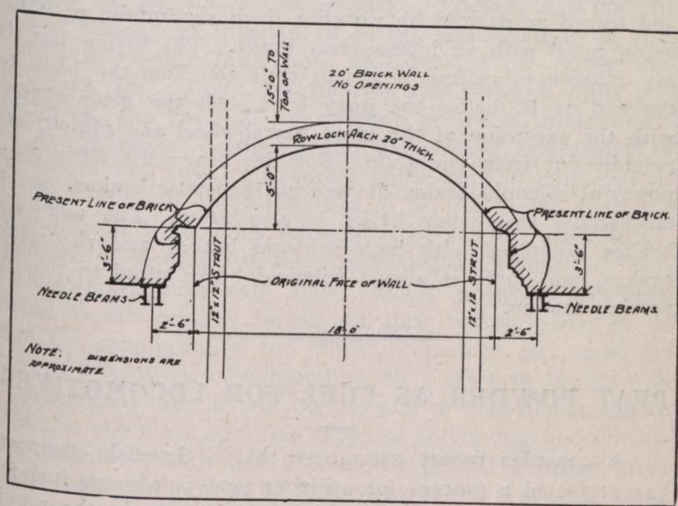


Fig. 1.

Part of the new work to be installed consists in placing steel columns in this wall, and in order to do this, the contractor has braced up part of the roof of the main building resting on the arch with long 12-in. by 12-in. strut timbers extending to the main floor of the church, has run needle beams through the wall at various points, two sets of which are shown in Fig. 1, and then has cut away the wall under the needle beams, and also under the ends of the brick arch. In Fig. 1, the original face of the wall is indicated, and the present line of the brickwork shown in a heavy line.

Why this arch stands, is something of a mystery, but the explanation, as far as any can be given, is as follows: The brickwork above the arch is acting as a false arch and, aided by the temporary struts supporting the roof, is carrying the load above, the true arch really being supported by this false arch, and the bricks of the arch are held in position solely by the adhesion of the mortar. It is a question in the writer's mind how far such a risk should be taken, with a span of this size, even with first-class brickwork. He strongly condemns this particular case, and, indeed, expects any day to pass by and find this arch in ruins. It is such work as this that, when a failure occurs, brings engineers and engineering into disrepute, although if the facts were known, it would probably be found that no engineer had anything whatever to do with this particular piece of work.

J. H. GANDOLFO,
Assoc. M. Am. Soc. C.E.

New York City, August 7, 1913.

CUTS IN NEWLY PAVED STREETS.

By James E. Barlow,
Assistant City Engineer, Cincinnati, Ohio.

THIS paper appears in the Proceedings of the American Road Builders' Association, and deals with the problem of underground services with which all cities have to contend. A cross section of a downtown city street shows car tracks, electric ducts, telephone conduits, water mains, gas mains, sewers, etc., and, of course, leading from these to the different buildings, are the necessary service branches. These all constitute a network which honeycombs pavement foundations, and it is the repair, renewal and extension of these which give rise to the problem which forms the subject of Mr. Barlow's paper.

It is a matter of common observation that an opening repaired is never as good as the original undisturbed pavement. In addition to the general disturbance of the subsoil there is usually left a slight hummock or a slight depression in the pavement which the pounding of traffic accentuates, causing a more or less endless chain of repairs. Eminent authorities have estimated that in some cities the useful life of the pavement is decreased 25 per cent. by these cuts. The reduction in the life of the pavement in a city such as Cincinnati is probably not as great as this, but nevertheless, the damage is very real. Some idea of this damage in dollars and cents may be had when one realizes that the cost of pavements, including maintenance, interest and renewals in a modern city like Cincinnati, exceeds $2,000,000 per year, i.e., if our pavements were laid and maintained by a public service corporation, a fair annual rental would exceed the figure mentioned. Now, aside from shortening the life of the pavement, these openings cause danger and annoyance to traffic, wear and tear to vehicles; they render cleaning more difficult, they are unsightly in themselves until repaired and they are costly to restore and they later become the source of additional repairs. Thus the importance of properly controlling such openings and reducing them to a minimum is really seen.

As a complete solution of this problem some have suggested subways or pipe galleries under the streets with laterals to property lines. Others have suggested subways under the sidewalks, necessitating double lines of mains. However, the investments tied up in the present form of underground construction, the enormous expense necessary for subways and the fact that much of the sidewalk space where subways are most needed is used privately for buildings as areaways, all tend to throw this solution, for most cities, into the somewhat distant future. The problem thus becomes one of best controlling the present cuts and reducing the number to a minimum.

When a street is to be improved notices of the two final steps in the legislation are sent to all the public service corporations, and again when bids are received, and again when the contractor is ordered to begin work. The object of these notices, of course, is to keep the companies informed of the status of the proposed improvement and allow them ample time to plan repairs, renewals or extensions. At the outset, however, it may as well be admitted that the franchises of these local public service corporations are such that we have little or no control over what they shall do along these lines.

As an additional lever, early this year a city ordinance was passed which virtually prevents any opening in a newly laid pavement for a period of three years, except in regular emergencies. Notices calling attention to this ordinance are sent to all public service corporations and to property holders about three months before the actual work of construction begins.

The topography of Cincinnati is such that often the heavy cuts and fills practically prohibit the laying of mains until the street is graded. On all contacts now let, if the street does not already contain water and sewer mains, they are included in the street improvement contract. We only wish we might include gas in the same way.

To prevent, in so far as possible, openings for house connections, we are now serving notices on all property owners along a proposed improvement to make water and sewer connections to their lots, whether occupied or vacant. If these are not made in 20 days, the city makes such connections to the curb line and assesses the cost thereof against the individual property. This calls forth considerable objection, but it must be borne in mind that half the owners gladly make all connections and it is only just, that we should take such steps to prevent the other half from damaging the pavement.

The preceding may all be classed as preventive measures. There is also to be considered the proper control and restoration of the necessary openings.

An important step in an intelligent control of street openings is to know what you have under the pavements and where these structures are. This primary information is not always in the hands of cities. In connection with an investigation for a comprehensive sewer layout, a complete underground survey is being made to locate all of the subsurface structures. These plots will be made on a scale of 40 and 50 ft. to the inch. There will be possibly 1,000 sheets, 24 by 34 ins., in the complete set. When compiled these will be kept up-to-date as a basis for our underground records and we can then more intelligently direct future underground construction.

There is, of course, also to be considered the actual making of the restorations. Several years ago a city ordinance was passed making it unlawful for any person other than the street repair department to open any pavement without a permit therefor. This ordinance further provided that the application for this permit should state the location and size of the opening and the kind of pavement. Before the permit is issued a deposit is required, based on the published sliding scale of prices for the restoration of different sized openings in the various kinds of pavements. No persons other than the employees of the street repair department are allowed to permanently restore these openings.

The party making the opening does the backfilling and makes a temporary restoration of the pavement; if the backfilling is not properly done the street repair department does it over and bills it to the party responsible.

The street repair department handles the issuing of all permits and, of course, makes all rules in accordance with which such openings can be made. The control of the openings and the responsibility for their speedy and adequate restoration is thus centralized.

This outlines the principal steps taken here to control street openings. Some of these measures cause much friction and delay, especially those involving serving of legal notices which require definite periods of time before action can be taken. However, it is felt that the end justifies the means.

SIXTY-TON ELECTRIC CANTILEVER CRANE.

A large cantilever crane constructed for loading blocks for building the harbor at Valparaiso was described in a recent issue of Engineering (London). The crane lifts the full load of 60 tons at 52 ft. from the centre of the crane, and is intended for raising the blocks from the railway trucks and transporting them to a barge lying alongside the jetty. The load is carried on a travelling jenny, and the machine is fitted with three independent lifts—viz., one lift for the maximum load of 60 tons operated by a 40-brake-horse-power motor, and two separate lifting motions, each capable of dealing with loads of 7½ tons, and operated by two 20-brake-horse-power motors. This arrangement was fitted so that the crane could deal with tipping-skips, the intention being that the skip and its contents, weighing 15 tons, might be lifted by the two 7½-ton motions operating synchronously and tipped as desired by working them separately, each lift being fitted with an independent brake. The lifting barrels are capable of coiling sufficient rope to allow the hook to descend 25 ft. below the quay level. All the gear wheels, with the exception of the large barrel-wheel and pinion, are machine-cut from the solid. For the heavy lift there is a powerful solenoid brake, as well as rheostatic brakes, and a mechanical emergency brake is also fitted. The auxiliary brakes are fitted with both solenoid brakes and rheostatic brakes, and there is also a solenoid brake fitted to the reversing motion.

PEAT POWDER AS FUEL FOR LOCOMOTIVES.

A consular report announces that a Swedish engineer, has perfected a process for utilizing peat powder as fuel for locomotives. The news has awakened interest in the possibility of developing the extensive peat bogs of Sweden.

The powder is manufactured by the Ekelund process. A factory has been in operation for several years in Sweden, under the management of the inventor of the powder process. It does not appear that the Ekelund process has made much headway as yet, but it is now predicted that in connection with the new discovery, the use of peat powder will in time become extensive.

In this system the peat powder is fed by an automatic process into the furnace of the locomotive, which is especially arranged to consume it. The Ekelund process is on the market in various countries, including the United States, but little has been made public concerning the new method. According to Mr. Von Porat, the inventor, the results obtained with peat powder may be summed up as follows:—

Substantially the same results can be had from 1½ tons of peat powder that one ton of coal will produce. Peat powder may be burned with an admixture of about 5 per cent. of coal. As to firing with peat powder, the work is almost nothing in comparison with firing with coal, because the powder is forced into the furnace by automatic process. No change had to be made in the boiler and none in the firebox, except installing the special apparatus. There is no difficulty in bringing the powder from the tender to the firebox, as it passes through a conveyance pipe. Another advantage in using peat powder is that no cold air can get into the firebox and neither smoke nor sparks escape from the smoke-stack.

As a result of engineer Von Porat's invention, it is reported that a number of the Swedish railways are preparing to use peat powder instead of coal.

MECHANICAL HANDLING OF MATERIALS

CLASSIFICATION OF APPARATUS — FLIGHT CONVEYERS — THE FIRST OF A SERIES OF ARTICLES ON INSTALLATION, MAINTENANCE AND OPERATION OF CONVEYING MACHINERY.

By REGINALD TRAUTSCHOLD, M.E.

CONVEYING and elevating machinery, though indispensable in almost any modern industrial operation or enterprise, is of comparatively recent development; the manufacture of such apparatus not commencing much before the beginning of the last quarter of the nineteenth century. To-day, fuel for the power house, the disposal of ashes, the handling of materials in cement mills and of ores at the mines and smelters, etc., raw materials to and through mills and factories, the handling of finished products, distribution of merchandise, dredging and excavating operations of all kinds, etc., etc., all require some kind of conveyer or elevator if such operations are to be performed with the economy and efficiency demanded by modern conditions. Though apparatus for the mechanical handling of materials is comparatively new, much progress has been made in this branch during the past twenty years or so and the requirements for power, capacity, etc., and the cost of operation, of equipment, etc., are now pretty well standardized, and it will be the aim of this discussion to describe the more common types of conveying and elevating machinery with such data as will permit a conservative and reliable opinion to be formed as to the economic value of each class of apparatus. Tables, charts and formulæ will be interposed wherever practical and illustrations of typical installations inserted throughout the text. The tables and charts, giving costs of apparatus, operation, maintenance, etc., will be conservative and refer to *good practice* in the Dominion. Their reliability will therefore depend on the proper care and attention to all machinery, for the results attainable can only be realized with apparatus that is not subject to abuse or neglect.

Conveying machinery may for convenience be distinguished from elevating machinery by considering the first to include all apparatus for carrying material along a horizontal plane or raising it from one plane to another through a comparatively gradual incline, and the second all apparatus that elevates material in a vertical plane or raises it through a comparatively steep incline. Naturally these two classes cannot be rigidly adhered to for much machinery for the mechanical handling of materials performs both operations of conveying and elevating. However, the division will facilitate a comprehensive discussion. Conveying machinery may be again subdivided into three general classes: 1st, apparatus that pushes or pulls the load, the moving parts not carrying the weight of the load; 2nd, apparatus that carries the load on moving parts; and 3rd, conveyers which depend for their operation on other than mechanical forces. Elevating machinery is classified in only the two last subdivisions for elevating operations by apparatus that drags or pushes the load are performed by conveyers at comparatively slight inclination, pumps not being considered in this discussion of mechanical handling of materials.

Flight (Scraper) Conveyers.—The oldest type of conveyer and one that is in common use to-day, in somewhat modernized form, consists of an endless chain to which are attached at intervals plates, or flights, which scrape the material handled along a trough in which are gates through which the load is discharged where desired. These conveyers may have simply one chain attached at the centre of the flight or else have two chains, one attached to each end of the flight. In the latter type of flight conveyer—double chain conveyer—the flights are usually equipped with wearing shoes that slide along guides on the sides of the conveyer and carry the weight of the moving parts preventing the flights themselves from coming in contact with and dragging along the trough, or else the flights are furnished with small wheels, or rollers, that run on rails or guides on the sides of the trough and perform the same functions as the shoes in the cruder type. Naturally the roller flight conveyer consumes rather less power than does a shoe flight conveyer under similar service, but the roller flight conveyer is somewhat more expensive. The balance of apparatus required for a flight conveyer consists simply of the trough, which is usually of steel, or some other material that will resist abrasive wear or of wood lined with a suitable wear-resisting metal; sprockets over which the conveyer chain runs; suitable driving mechanism, consisting usually of a system of gears by means of which the speed of the power unit is reduced to the proper amount; the gates and chutes through which the material is discharged from the conveyer, and chutes for loading the conveyer.

Three conditions govern the capacity of flight conveyers—the weight of the material to be conveyed being neglected—1st, the speed at which the conveyer is run; 2nd, the area of the flights presented against the material dragged along the trough; and 3rd, the distance between flights (intervals). The advisable speed of conveyer depends largely upon the nature of the material to be conveyed and may be anywhere from 100 to 200 feet per minute. The flights are usually rectangular in shape with a length (slightly less than the width of the trough) about three times as great as their width (the dimension at right angles to the plane of the trough) and are spaced from one to two feet apart. Two other conditions effect the capacity of this type of conveyer, however, which depend upon the nature of the material conveyed more than upon the conveyer itself. They are, the weight of the material and its angle of repose. The first effects the capacity of the conveyer directly; that is, the capacity of a given conveyer at a given speed varies directly with the weight of the material handled. The second condition has little effect on horizontal conveyers—the angle of repose of the average class of material handled by flight conveyers not varying to any very great extent—but does effect the capacity of inclined conveyers. For all practical purposes, however, the reduction in capacity of an inclined flight conveyer depends directly upon the inclination of the conveyer. For instance, the capacity of a flight conveyer inclined 10 degrees to the horizontal is but about 80 per cent. of that of a similar horizontal conveyer; a conveyer inclined 20 degrees, about 66 per cent.; while a flight conveyer, inclined 30 degrees to the horizontal, about the limiting inclination for such a conveyer, would have a capacity about one-half as great as that of a conveyer running in a horizontal plane. Flight conveyers carrying a load from one elevation to a lower one has increased carrying capacity, the limit being that point at which the angle of repose of the material is equal to the angle of declination. These latter conveyers, those running down hill, are so rare, however, that they may be overlooked in this discussion, any such installation being quite special and presenting a distinct engineering problem. Flight conveyers, though depending directly upon the weight of the material handled for their capacity, should not always be run at the same speed nor is there any other rule but experience for ascertaining the most advisable speed of conveyer. Efficient speeds for certain materials handled by this type of conveyer are given in Table I., together with the average weight of such materials per cubic foot. Table II. gives the capacity of some of the common sizes (width x length of flights) of horizontal flight conveyers carrying material weighing 100 pounds per cubic foot at a speed of 100 feet per minute, conveyer being uniformly and continuously loaded. The capacity of similar conveyers carrying material of other weight and at different speed varies directly with both the weight and speed. For instance, a conveyer carrying material weighing 50 pounds per cubic foot at a speed of 200 feet per minute would have a capacity the same at that given in Table II., conveyer handling material weighing 100 pounds at a speed of 100 feet per minute. Inclined conveyers have their capacity reduced from 1⅔ to 2 per cent. for each degree of angle of inclination. The approximate capacity of any flight conveyer may also be ascertained through the use of formulæ. (See Formulæ I. and II.).

Table I.—Economic Speeds of Flight Conveyers for Various Materials.

Material	Weight per cubic foot	Advisable Speed
Coke.............	32 to 35 pounds	100 ft. per min.
Broken Stone (course)....	160 to 170 pounds	125 "
Lump Coal—R/M	50 to 60 pounds	125 "
Ashes	About 45 pounds	150 "
Lime and Cement	50 to 80 pounds	150 "
Ore	About 125 pounds	175 "
Crushed Stone	150 to 170 pounds	175 "
Sand and Gravel	100 to 120 pounds	175 "
Fine Coal	About 50 pounds	200 "

Table II.—Capacity of Flight or Scraper Conveyers.

Horizontal Conveyers.

Capacity in tons per hour at a conveyer speed of 100 feet per minute. Material weighing 100 pounds per cubic foot.

Size of Flights	Flights Spaced 12″	16″	18″	24″
4″ x 10″	90	67	60	45
4″ x 12″	114	85	76	57
5″ x 12″	138	103	92	69
5″ x 15″	186	139	124	93
6″ x 18″	240	180	160	120
8″ x 18″	360	270	240	180
8″ x 20″	420	315	280	210
8″ x 24″	540	405	360	270

$$W_h = \frac{0.0002443\ AVw'}{S} \qquad \text{Formula I.}$$

$$W_i = W_h - \frac{0.016285\ L'W_h}{S} \qquad \text{Formula II.}$$

Where:—

W_h = Weight of maximum load in tons per hour—Horizontal Conveyer.

W_i = Weight of maximum load in tons per hour—Inclined Conveyer.

V = Velocity (speed) of Conveyer in feet per minute.

A = Area of Flights in square inches (width x length).

S = Spacing of Flights (intervals) in feet.

W′ = Weight of material conveyed in pounds per cubic foot.

L′ = Inclination of Conveyer in degrees—Inclined Conveyer.

The consumption of power in running a flight conveyer or any other similar piece of mechanism may be divided into that required to run the conveyer itself and that required to drag the load. In the case of flight conveyers, however, the latter operation is so much more severe that the power required to run the empty conveyer is but a comparatively small percentage of that required to run one fully loaded. The power required for moving the load depends upon the load handled and is relatively so great, compared with the requirements of the unloaded conveyer, that the latter may be taken, for all practical purposes, as also depending upon the load handled. This simplification of the power question has led certain manufacturers of such equipment customarily to figure the power requirements of conveyers with shoe flights and those with roller flights as identical. These two refinements of the old-fashioned drag conveyer have considerably reduced the power consumption of such conveyers, one to an appreciably greater extent than the other, however, and this fact is taken into account in the derivation of the following formula for ascertaining the horse-power required properly to drive a flight conveyer that is continuously and uniformly loaded—Formula III. Conveyers without shoe or roller flights are now limited to small sizes and to those that are relatively short, so that, while Formula III. is not exactly applicable to such conveyers, the error (lack of power) is so slight that the formula may be used by making an allowance of small percentage in cases of conveyers without the refinements of shoes or rollers. In fact, the choice of the power unit depends so largely upon the sizes of standard motors, engines, etc., that the most convenient size of power unit has usually an abundance of power to care for the slight error. For instance, a conveyer requiring 8½ horse-power would probably be equipped with a 10 horse-power motor which would have a capacity more than sufficient to care for any additional power required. The use of Chart I., which of necessity cannot be read as accurately as the results attainable by the use of the formula, will obviate the necessity of making any such corrections, the chart readings being sufficiently conservative to cover any small error.

Horsepower:—

W = Weight of load conveyed in tons per hour.

V = Velocity (speed) of Conveyer in feet per minute.

H = Height to which load is elevated in feet.

W_c = Weight of moving parts of Conveyer in pounds per foot (average) = 0.74 W—Conveyers with shoe flights.
0.92 W—Conveyers with roller flights.

W′ = Weight of load conveyed per minute per foot in pounds $= \frac{33W}{V}$

f_s = Speed factor = 0.18—from experiment—shoe flights.
= 0.09—from experiment—roller flights.

f_l = Load factor = 0.7—from experiment.

Horsepower required to run Conveyer empty.

$$HP = \frac{\left.\begin{matrix}0.74\ W\\0.92\ W\end{matrix}\right\} \times \frac{33W}{V} \times \left.\begin{matrix}0.18\\0.09\end{matrix}\right\} \times V \times L}{33000}$$

$$= \frac{0.1332\ WL}{1000} \quad \text{(Conveyers with shoe flights.)}$$

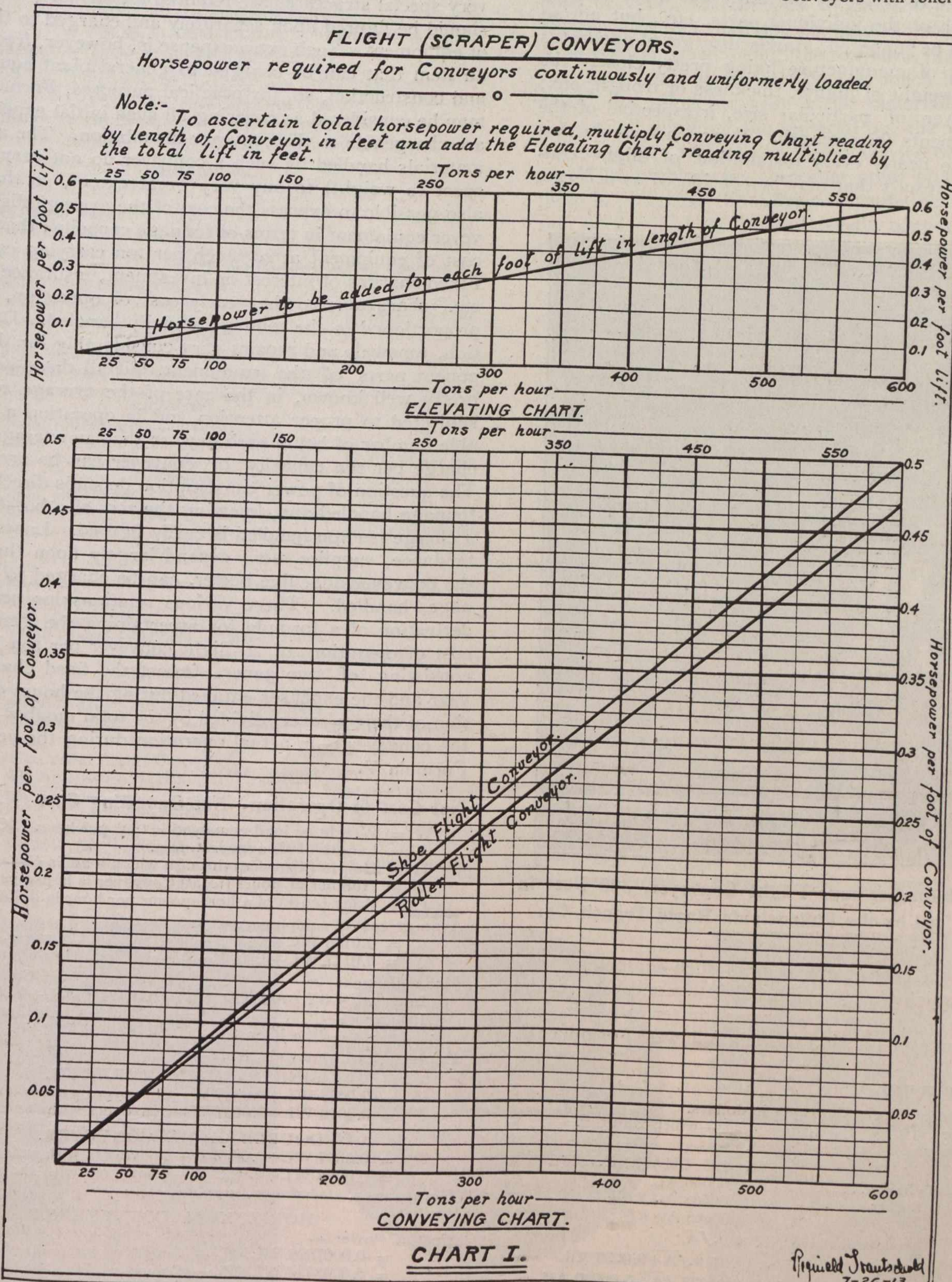
$$= \frac{0.0828\ WL}{1000} \quad \text{(Conveyers with roller flights.)}$$

Total horsepower required.

$$HP = \frac{C\ WL}{1000} + \frac{WH}{1000} \qquad \text{Formula III.}$$

Where :—

C = Constant = 0.8332—Conveyers with shoe flights.
= 0.7828—Conveyers with roller flights.



Horsepower required to convey load.

$$HP = \frac{W' \times 0.7 \times V \times L}{33000} = \frac{0.7\ WL}{1000}$$

Horsepower required to elevate load.

$$HP = \frac{W' \times V \times H}{33000} = \frac{WH}{1000}$$

As the total weight of a well designed and proportioned flight conveyer bears a close relationship to its capacity, and as the cost of such a conveyer also varies nearly directly with the weight of the apparatus, it is possible to form a very close approximation of the cost of the ordinary flight conveyer in terms of its capacity; that is, in terms of size and length of conveyer. The area of the flights governs largely the capacity of any flight conveyer, the strength of the chains required, sizes of sprockets, power of drive, etc., and consequently governs the cost of these component parts. The load handled by a particular size of conveyer naturally has some bearing on the strength of the individual parts, etc., but not so much as would be supposed; standardization of apparatus and economies of manufacture fixing pretty closely the size of chain, weight of flights, thickness of trough, etc., for any conveyer of particular size, irrespective of the actual requirements for strength. Hence, a formula that is applicable to nearly every installation of flight conveyers, giving results that may be considered as accurate

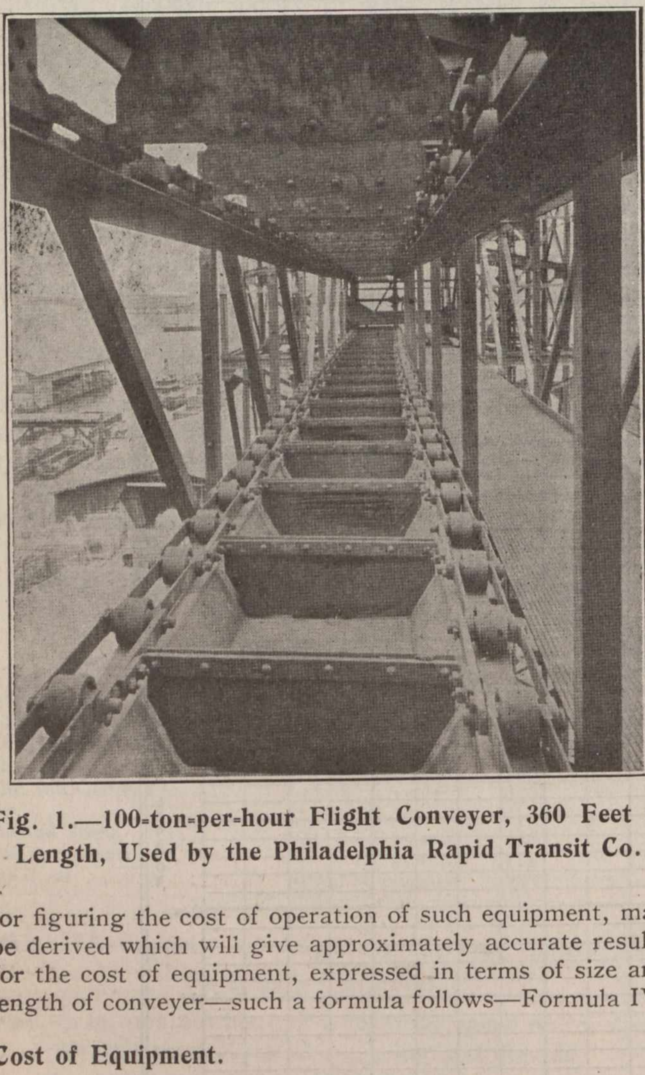


Fig. 1.—100-ton-per-hour Flight Conveyer, 360 Feet in Length, Used by the Philadelphia Rapid Transit Co.

for figuring the cost of operation of such equipment, may be derived which will give approximately accurate results for the cost of equipment, expressed in terms of size and length of conveyer—such a formula follows—Formula IV.

Cost of Equipment.

C = Cost of Flight Conveyer in dollars.
A = Area of flights in square inches—width x length.
L = Length of Conveyer in feet.

SINGLE CHAIN CONVEYER

Average cost of chains	$0.00066\ AL$
flights (roller) ..	$(0.00568\ A + 3.0)L$
(shoe)....	$(0.00568\ A + 1.5)L$
sprockets, drive, &c.	$6.0\sqrt{A}$
trough and rails, &c.	$(0.0413\sqrt{A} + 0.0039\ A)L$
gates, &c.	$(0.01033\sqrt{A} + 0.00089\ A)L$

ADDITIONAL—DOUBLE CHAIN CONVEYER

Chain Attachments, &c.	$0.000163\ AL$
Sprockets, &c.	$1.0\sqrt{A}$

Then:—

$$C = (DA + 0.05163\sqrt{A} + E)L + F\sqrt{A} \qquad \text{Formula IV.}$$

Where:—
D = Constant = 0.01113—Single Chain Conveyer.
= 0.011293—Double Chain Conveyer.
E = Constant = 3.0—Conveyers with roller flights.
= 1.5—Conveyers with shoe flights.
F = Constant = 6.0—Single Chain Conveyer.
= 7.0—Double Chain Conveyer.

The cost of equipment as given by Formula IV., covers the cost of erection of a simple installation, but, if very special structures are required for support, etc., they should be figured upon separately and charged to the cost of equipment. Such extra expense is, however, frequently an item that can be charged to general plant equipment and construction, so, for practical purposes, Formula IV. may be considered as covering all such initial expenses as are chargeable to the conveyer installation. The average materials handled by such a conveyer do not vary (in respect to weight) to any very great extent, so that it is also possible to express the cost of the average flight conveyer equipment in terms of tonnage capacity; that is, the cost of equipment at so much per ton carrying capacity. Fixed charges of interest on investment, insurance, taxes, etc., chargeable to the yearly cost of operation, can be proportioned to the tonnage actually handled. Depreciation, renewals and repairs vary considerably for the component parts of the equipment, but all these items are pretty well known, in the case of the average conveyer subjected to proper attention and in operation a reasonable number of hours each day, so that an average yearly charge per ton capacity of conveyer can be arrived at. The question of power consumption depends directly upon tonnage handled and, knowing the cost of a horse-power, a charge per ton handled is easily figured. Labor, or attendance, supplies, etc., depend largely upon the use of the conveyer and, like power, can be charged to the tonnage handled. These various relationships permit the derivation of a formula for ascertaining the average net cost of operation of a flight conveyer per ton handled, consisting of two general items, the fixed charges per year and the expenses entailed during the hours of operation of the conveyer, divided by the total number of hours the conveyer is in actual operation during the year. See Formula V.

Net Cost of Operation—Net Operating Cost (N.O.C.).

W = Weight of load conveyed in tons per hour. (Capacity.)
L = Length of Conveyer in feet.
H = Height (distance) through which load is elevated in feet.
N = Number of hours (total) Conveyer is in use per year.
P_c = Price (cost) of a horsepower per hour in dollars.

Average cost of equipment:—
= $0.0146\ WL + 0.75\ W$—Single Chain Conveyer.
= $0.0147\ WL + 0.85\ W$—Double Chain Conveyer.

Fixed charges:—

Interest —6% total cost		Single Chain Conveyer
Insurance—1%	=	$0.0012410\ WL + 0.06375\ W$
Taxes —2%—¾ cost		Double Chain Conveyer
		$0.0012495\ WL + 0.07225\ W$

Depreciations, renewals, &c.:—

SINGLE CHAIN CONVEYER.

= $0.0035\ WL + 0.0807\ W$—Conveyers with roller flights.
= $0.0040\ WL + 0.0807\ W$—Conveyers with shoe flights.

DOUBLE CHAIN CONVEYER.

= $0.0037\ WL + 0.0922\ W$—Conveyers with roller flights.
= $0.0038\ WL + 0.0922\ W$—Conveyers with shoe flights.

Horsepower, attendance, supplies, etc.:—

HORIZONTAL CONVEYERS.

Cost of Power:—
= $0.0007828\ WLNP_c$ — Conveyer with roller flights.
= $0.0008332\ WLNP_c$ — Conveyer with shoe flights.

Cost of labor or attendance:—
= $0.000025\ WLN$ — average.

Cost of supplies, &c.:—
= $0.0000676\ WLN$ — Conveyers with roller flights.
= $0.0000909\ WLN$ — Conveyers with shoe flights.

INCLINED CONVEYERS.

Cost of extra power:— = $0.00101\ WHNP_c$ — average.
Cost of extra labor, &c.:—Negligible.
Cost of extra supplies, &c.:— = $0.0000133\ WHN$ — average.

Then :—

Net Operating Cost per ton :—

$$\text{N.O.C.} = \frac{AL+B+CLN+1.33\,HN+(DL+101\,H)\,NP_c}{100{,}000\,N} \qquad \text{Formula V.}$$

Where :—

A = Constant = 474.10 — Single Chain Conveyer—roller flights.
= 524.10 — Single Chain Conveyer—shoe flights.
= 494.95 — Double Chain Conveyer—roller flights.
= 504.95 — Double Chain Conveyer—shoe flights.

B = Constant = 14445 — Single Chain Conveyer.
= 16447 — Double Chain Conveyer.

C = Constant = 9.26 — Conveyers with roller flights.
= 11.59 — Conveyers with shoe flights.

D = Constant = 78.28 — Conveyers with roller flights.
= 83.32 — Conveyers with shoe flights.

Examples.

Conditions :—

Length of Conveyer.... 100′ 0″ = L
Material elevated ...... 20′ 0″ = H
Service................ 2400 hours per year = N
Cost of power.......... $0.02 per horsepower per hour = P_c

Single Chain, roller flight, Conveyer.

$$\text{N.O.C.} = \frac{47410+14445+2222400+63840+472704}{240000000} = \$0.01175 \text{ per ton.}$$

Single Chain, shoe flight, Conveyer.

$$\text{N.O.C.} = \frac{52410+14445+2781600+63840+496896}{240000000} = \$0.01421 \text{ per ton.}$$

Double Chain, roller flight, Conveyer.

$$\text{N.O.C.} = \frac{49495+16447+2222400+63840+472704}{240000000} = \$0.01176 \text{ per ton.}$$

Double Chain, shoe flight, Conveyer.

$$\text{N.O.C.} = \frac{50495+16447+2781600+63840+496896}{240000000} = \$0.01421 \text{ per ton.}$$

Conditions :—

Same as in preceding example except that service is but 1200 hours per year.

Single Chain, roller flight, Conveyer.

$$\text{N.O.C.} = \frac{47410+14445+1111200+31920+236352}{120000000} = \$0.01201 \text{ per ton.}$$

Single Chain, shoe flight, Conveyer.

$$\text{N.O.C.} = \frac{52410+14445+1390800+31920+248448}{120000000} = \$0.01448 \text{ per ton.}$$

Double Chain, roller flight, Conveyer.

$$\text{N.O.C.} = \frac{49495+16447+1111200+31920+236352}{120000000} = \$0.01204 \text{ per ton.}$$

Double Chain, shoe flight, Conveyer.

$$\text{N.O.C.} = \frac{50495+16447+1390800+31920+248448}{120000000} = \$0.01448 \text{ per ton.}$$

The use of Formula V. gives results that agree very closely with those attained in ordinary good practice, and it is interesting to note that the economic value of the conveyer does not vary greatly with service, if subjected to reasonable usage. In the examples cited—the conditions of which are common in practice—there is only a difference of less than 2½ per cent. between the net cost of operation when using the conveyer 2,400 hours per year and 1,200 hours. Of course, much less than 1,200 hours' operation per year would materially increase the net cost of handling the material but the value of an installation of which such slight service only is required would be problematic. Another point of interest is that there is practically no difference in the net cost of operation of a one-chain or a two-chain conveyer—indicating the correctness or accuracy of the formulæ—although the refinement of roller flights does considerably reduce the cost of handling, the reduction of about 17 per cent. indicated by the formulæ being confirmed in practice as a fair record of the difference in actual operating costs of the two types of flight conveyers. The choice between a one and a two-chain conveyer is governed almost exclusively by the size of the conveyer—the larger conveyers usually having two chains. Severe conditions, however, under which operation must be performed sometimes makes it advisable to equip even the small sizes of conveyers with the two chains. In such a case, the initial cost of equipment would be somewhat increased in order to obtain satisfactory wear and operation of the conveyer and Formula V. would be applicable even under such conditions.

The foregoing discussion has been limited to the ordinary type of flight conveyer as found in the average mine or industrial plant, but many other adaptations and modifications of this standard type of conveyer are in common use; such as log hauls, ice conveyers and crude drag conveyers of all descriptions, which present many problems that require individual solution that cannot be taken up in the present discussion. The problems presented in these special cases are not, however, difficult of solution and modifications of the formulæ that have been given will be found to meet the requirements of almost any installation of flight or drag conveyers.

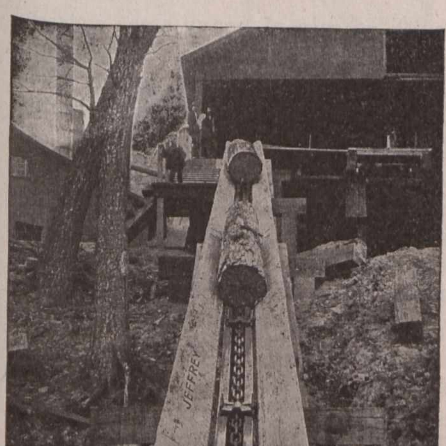


Fig. 2.—Log Haul.

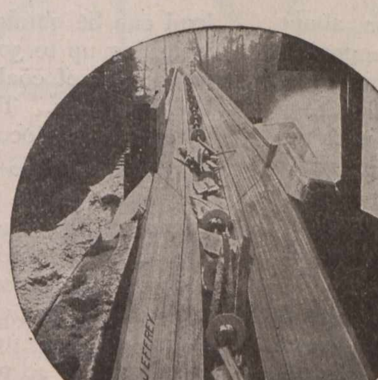


Fig. 3.—Wire Rope Offal Conveyer.

Fig. 4.—Refuse Conveyer.

Drainage work in Manitoba made good progress during 1912, according to the report of the Department of Public Works for the fiscal year ending November 30th, 1912. At that date 1,844.8 miles of drains had been constructed, benefiting in the neighborhood of 1,984,000 acres.

THE BOUCANNE RIVER VIADUCT

GENERAL FEATURES OF THE DESIGN OF THIS NATIONAL TRANSCONTINENTAL RAILWAY VIADUCT—OUT-OF-THE-ORDINARY METHODS OF ERECTION — PAPER PREPARED FOR INSTITUTION OF CIVIL ENGINEERS, GREAT BRITAIN.

By P. L. PRATLEY, M.E., Assoc. M. Inst. C.E.
Designing Engineer, Dominion Bridge Co., Limited, Montreal

THE National Transcontinental Railway crosses the Boucanne River at a point 167.2 miles east of the Quebec Bridge, which is the St. Lawrence River crossing of the same line. Both crossings are situated in the province of Quebec, in the B district of the eastern division of the railway.

The Boucanne River crossing occurs just north of the peak of Maine, where this State geographically juts into British territory. Although the river at this point is but a shallow stream with a depth varying with the seasons from a few inches to 5 feet, and a width of about 100 feet, the valley in which it flows is quite a considerable depression. These conditions, together with the fact that the line follows generally the the "height of land," necessitate a crossing 790 feet in length, with a maximum height of 127 feet from the base of the rails to water-level. The viaduct is of the trestle type, and as indicated by the general outline in Fig. 1, it consists of five towers of 30 feet span and two of 40 feet, with intermediate spans of various lengths.

of course, affected by the vertical spiral at the west end whereby the superelevation of the 6° curve is gradually attained, but not to any larger extent than can be adjusted by the size of the ties used on the end span. The necessary tilting of the track on the bridge is also small enough to be effected in the ties.

The ground at the side is principally earth and boulders, with solid rock about 10 feet below, so that the concrete pedestals and abutment-walls were well founded on the bed rock.

Points in the Design.—The viaduct is designed in accordance with the specifications issued by the Dominion Government in 1908, with one or two further restrictions imposed by the bridge-engineers of the National Transcontinental Railway, which will be specifically alluded to later. The steel was proportioned for a live load known as Class "Heavy." This consists of two engines with tenders, the whole weighing 180 (short) tons, on a combined wheel-base of 104 feet, followed by a uniform train-load of 4,750 lbs. per lineal foot of track. This uniform

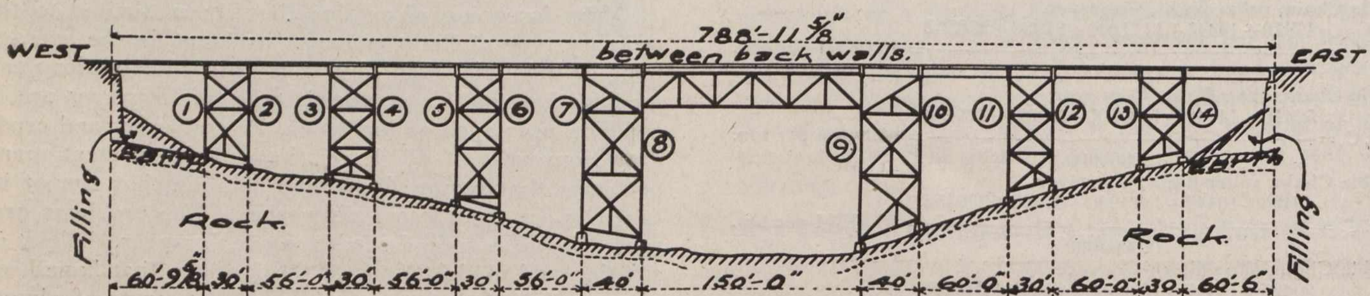


Fig. 1.—General Outline of Viaduct.

The viaduct is perfectly straight, except for about 135 feet at the west end, where it lies on the approach spiral to a 6° curve (radius 977 feet), which commences 165 feet west of the bridge. The transition spiral is thus 300 feet long, and is laid out as a cubic parabola with the equation $y = kx^3$, y being the offset from the tangent, x the distance from the beginning of the spiral measured along it, and k a constant, in this case equal to 0.00000058232; so that at the end of the transition spiral, where $x = 300$, $y = 15.72$ feet.

The viaduct is of the deck type, that is, the track is laid on cross timbers or "ties" placed with 4-inch spaced between them. The ties are 10 inches wide by 12 inches deep, and are generally 14 feet long, "dapped" ½ inch over the top flanges of the girders, and held in place longitudinally by timbers running along the ends of the ties and "dapped" over them. The girders are 9 feet apart between centres, and are uniformly 6 feet deep. The transverse batter of the towers is 1 in 6, and this, together with the depth of the truss and its detail, brings the width of the truss span to 17 feet 2¼ inches between centres. The viaduct is also on a gradient of 1.10 per cent. (1 in 90), falling toward the east. The gradient is,

load can be obtained with a train of heavy steel wagons weighing up to 30 or 35 tons each, and each carrying up to 60 tons of coal, their length being 36 to 40 feet over the couplings. The maximum axle-load on the driving-wheels of the locomotive in this class is 49,400 lbs.

The specification calls for an extra allowance of load for impact equal to the figure obtained by dividing the square of the live load by the sum of the dead and live loads. In addition to this, for all spans of less than 80 feet, including the stringers and cross floor-beams of the truss span, the live load itself is increased by a percentage which depends upon the length of the span, having a maximum of 40 per cent. at the theoretical span of zero, diminishing according to the formula $0.40 - \frac{l}{200}$, where l is the length of the span in feet, and vanishing at 80 feet. In the Transcontinental Railway bridges this increment of live load is introduced into the live-load figure used in the impact formula, as well as into the pure live load, so that for the girders of the Boucanne River Viaduct the impact load was obtained by dividing the square of the increased live load by the sum of the increased live load and the dead load. The sum of the dead load, the increased live load, and the impact, thus obtained, is treated as the total load both for shear and bending moments. For this load a unit shear of 10,000 lbs. per square inch is allowed on the gross section of the webs of plate-web girders; and on the net section of the tension flange a unit fibre-stress of 16,000 lbs. per square inch is permitted.

Fig. 2.—View of the Completed Viaduct.

The gross area of the compression flange is made equal to the gross area of the tension flange, and 75 per cent. of the gross bending value of the web-plate is considered as effectively assisting the flanges to resist the induced bending-moments.

Under these specifications the weight of steel in the 60-foot deck plate-girder spans is 57,000 lbs. per span, including both girders and all wind- and sway-bracing. In the 56-foot spans the corresponding weight was 51,000 lbs., in the 40-foot spans 29,000 lbs., and in the 30-foot spans 17,500 lbs. per span.

The principal object of this paper is to describe the methods employed in the erection of the larger towers and the truss span. The trusses of the 150-foot span, as will be seen from Figs. 1 and 3, have six panels of 25 feet, are in vertical planes, measure 25 feet deep between the centres of gravity of the chords, and are of the Warren type. The cross girders or floor-beams at the intermediate panels are 4 feet 4¼ inches deep over the flange-angles and are riveted to the vertical members of the truss just under the top chord. The end floor-beams, which have to support the 40-foot girder spans, are 5 feet ¼ inch deep over the flange-angles, and are 1 foot lower on the truss than the intermediate beams, in order to allow for the difference in depth between the 40-foot girders mentioned and the stringers of the truss span. The latter are also plate-web girders, their depth thus being 5 feet ¼ inch from back to back of the angles against the uniform 6 feet ¼ inch of the girder spans. It will thus be seen that the stringer girders sit on top of the floor-beams and are 9 feet apart between centres, being in fact 25-foot deck plate-girder spans. The floor-beams are similarly single-web plate girders with webs 7/16 inch thick and flange-angles 6 inches by 6 inches by 9/16 inch in the deeper and 6 inches by 6 inches by ⅝ inch in the shallower. The material in the top chord is ½ inch throughout, while both angles and plates in the bottom chord are 7/16 inch in thickness in the first two panels at each end and ¾ inch thick in the two central panels. The web system of vertical and diagonal members is consistently of H shape, the plate being ⅜ inch thick, except in the compression diagonals, where ½-inch plate is used. The angles are 9-inch by 3½-inch by ½-inch bulb angles in all the diagonals and the end verticals, whilst in the verticals that are merely hangers they are 6-inch by 3½-inch by ⅜-inch angles, and in the intermediate compression verticals 7-inch by 3½-inch by ⅝-inch angles. The main or end compression diagonal is further reinforced by 19-inch by ¾-inch plates on the outside of each pair of bulb angles.

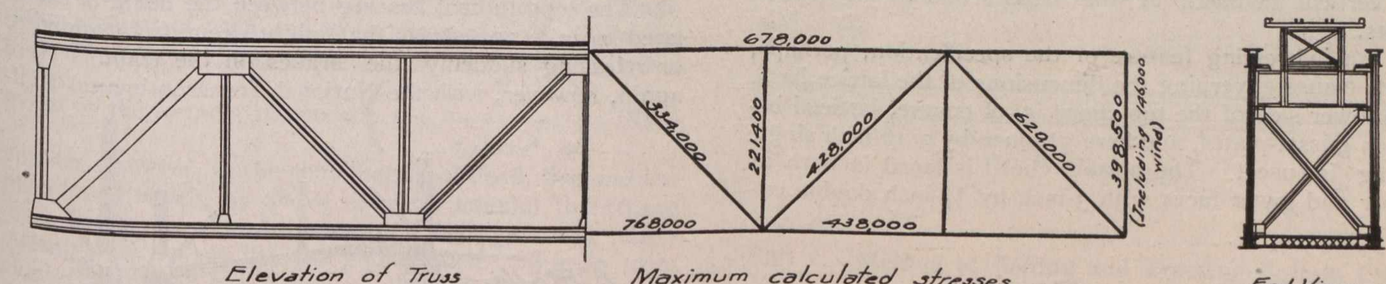


Fig. 3.—The 150-Foot Span Truss and Maximum Stresses.

The impact-formula explained above will be seen to be based on two main assumptions. The first is that the greater the dead load of a span of given length, the less will be the dynamic effect of the sudden application of the live load. For example, a pony-truss span carrying a ballast and concrete floor would suffer less impact load than girders carrying the usual timber deck. The second assumption is that the longer a span, the less likely is the maximum dynamic effect to occur with the maximum live load. The shorter the span, the more suddenly will the live load reach the maximum and the more severe will be the vibratory effects of unbalanced engine-wheels and jumping loads. On the other hand, the longer the span the more gradually does the induced stress attain its maximum, and the more generally distributed will be the effect of impinging loads.

The maximum stress in the top chord, composed of dead load, live load and impact load, amounts to 678,000 lbs., for which a section of 50.5 square inches is provided, the maximum unit compression allowed by the engineers of the Transcontinental Railway being 13,600 lbs. per square inch. This figure is an arbitrary restriction on the government specification, which would permit 14,600 lbs., the formula followed being 16,000 ÷ $\left(1 + \frac{l^2}{18{,}000\, r^2}\right)$, assuming fixed ends.

The bottom chord receives a possible maximum stress of 768,000 lbs., which at 16,000 lbs. per square inch requires 48 square inches of net sectional area. To meet this 48.76 square inches is supplied, being the effective net section obtained from the 60.76 square inches gross section provided. The Transcontinental Railway also prescribes that the Government formula for free-ended compression members shall be used for their vertical struts, whilst for the compression diagonals the intermediate formula $16{,}000 \div \left(1 + \frac{l^2}{12{,}000\, r^2}\right)$ is used, the constant before r^2 being 9,000 for free ends. By this means the units indicated in Fig. 3 are obtained, and by their aid the necessary cross section is calculated. The tension unit is 16,000 lbs. per square inch throughout.

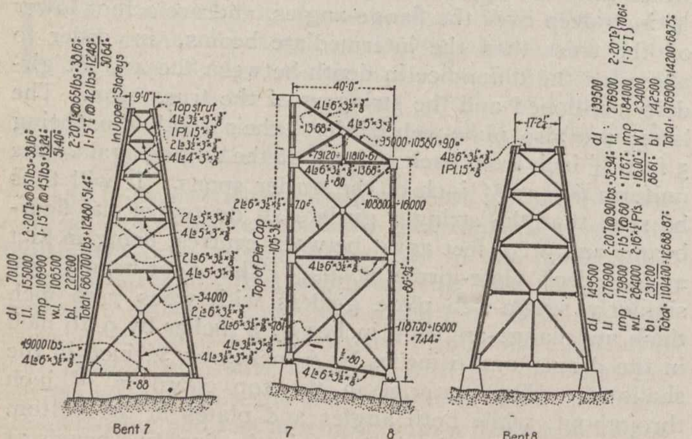


Fig. 4.—Typical Compression Stresses.

The trusses are braced together horizontally by both top and bottom lateral systems, the top consisting of crossed single diagonal angles in each panel, connected to the underside of the chord-angles and to the bottom flange of the stringers. The bottom lateral bracing consists of transverse horizontal struts of four angles laced in pairs and of cross diagonals similarly constructed. They are made the full depth of the chord, for the sake of simplicity and efficiency in connection. Undtr each floor-beam there is a sway-brace consisting of cross diagonals connected to the vertical members of the trusses and to the bottom struts.

An interesting feature of the specification, perhaps, is the clause governing the dimensions of the lattice bars. The lower side of the top chord is, of course, latticed between gusset-plates, and here 5½-inch by 9/16-inch single lacing is used. The bottom chord is laced in both its upper and lower faces with 3-inch by ½-inch double lacing, whilst the lacing of all the lateral struts and diagonals is 2½ inches by ⅜ inch. The specification requires that the thickness of the bars shall be at least 1/40 of the distance between the rivets that connect their ends to the two leaves of the members which they lace, if singly laced, and 1/60 of that distance if doubly laced. On the upper chord, the web-plates being 14¼ inches apart, the gauge-lines on the angles are 20¼ inches, which with approximately 60° lacing gives 23 inches between rivets and 23/40 inch or 0.575 inch as the required thickness, for which 9/16 inch was used. The width is governed by the fact that two rivets were required at each end, and these could not be obtained with the necessary edge distances on the lattice bar under 5½ inches. Similarly on the bottom chord, the same 20½ inches with 45° lacing gives 28½ inches between end rivets and calls for bars $\frac{28.5 \text{ inches}}{60}$ or 0.48 inch in thickness. One rivet being sufficient connection, a 3-inch bar was adopted here. The latticing in the bracing members need only be made 75 per cent. as heavy as the above specification, and so ⅜ inch is the usual dimension.

The stresses permitted in rivets are 10,000 lbs. per square inch in single shear and 20,000 lbs. per square inch bearing stress. These figures are for shop-driven rivets: for rivets driven by power in the field a reduction of 10 per cent., and for hand-driven rivets a reduction of 25 per cent. is prescribed. The gusset-plates in the main truss were principally ⅝ inch thick, and the minimum thickness of material allowed throughout the bridge was ⅜ inch.

The towers are of the usual viaduct construction, and consist of two "bents" braced together longitudinally. Each bent consists of two inclined posts with sway-bracing between them. As indicated in Fig. 4, the sway-bracing is made up of panels of crossed diagonals and horizontal struts. This is the type most usually adopted, although occasionally the struts are omitted except at the extreme top and bottom, and the diagonals are made rigid enough to resist compression effectively. In the usual design, typified by the Boucanne River Viaduct, the diagonals, although made of stiff shapes, are proportioned for tension with a limit of slenderness determined by making the maximum permissible ratio of unsupported length to radius of gyration equal to 200. The horizontal struts are then calculated as compression members, and their stiffness is governed in turn by a clause limiting l/r to 120. Under these conditions the choice of sections is governed more generally by the stiffness than by the calculated stress.

The longitudinal bracing between the bents is calculated to take any stress that might occur from applying or releasing suddenly the brakes on the train. Here again, however, with the National Transcontinental Railway, the same stiffness clauses operate: in the smaller towers they determine the dimensions of the diagonals and struts, whilst in the larger towers carrying the ends of long spans the calculated stresses often call for thicker metal. A typical diagram is given (Fig. 4) showing the stresses computed and the material supplied for the particular tower to be considered in this paper.

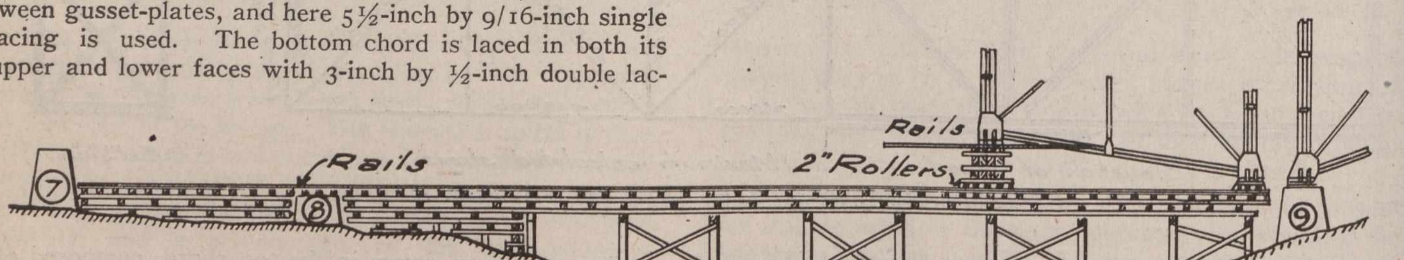


Fig. 5.—Staging Track for Moving Towers Into Place.

As shown in Fig. 4, the posts are made up of three **I**-beams riveted together to form an **H** section, reinforced by side plates where necessary. This form of section for posts entails the minimum of shop-work, and secures reasonable connections for all bracing. When the two **I**'s are deep, they may be connected at suitable intervals between bracing-points by batten-plates, or may be latticed continuously. In the case of the Boucanne Viaduct the batten-plate system was employed, and 9-inch by ⅜-inch plates 1 foot 3 inches long were provided on each face at intervals of about 5 feet. In the tower 7-8 under consideration the lowest story was built up of two 20-inch I-beams, each weighing 90 lbs. per lineal foot, with a diaphragm I-beam 15 inches deep and weighing 60 lbs. per

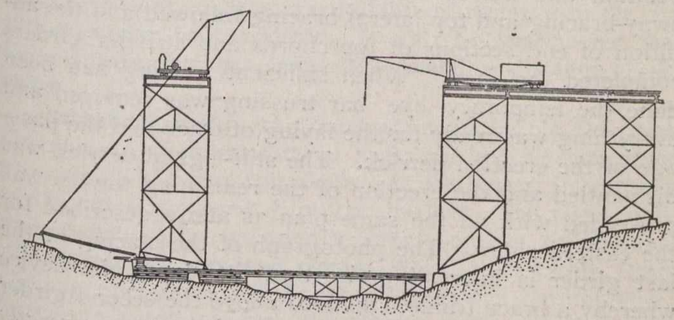


Fig. 6.—Method of Moving Tower.

foot. In addition, there were two 16-inch by ½-inch side plates, one on the outside of the web of each 20-inch beam, giving a total gross section of 86.6 square inches. The beams are of heavy American standard sections, and were rolled by the Carnegie Steel Co. The loads on this post, No. 8, are computed to be as follows:

Dead load	149,500
Live load	276,900
Impact	179,800
	606,200
Wind load	264,000
Brake load	231,200
Total	1,101,400

These loads are the maximum that can co-exist according to the specification and not necessarily the maximum amount of each and every class of load. Now the unit compressive stress allowed by the specification is that given by the formula $16,000 \div \left(1 + \frac{l^2}{9,000\, r^2}.\right)$ This applies, however, to the combination of dead, live and impact loads which, as above detailed, amount to 606,200 lbs. In the story under inspection $l/r = 72$ and consequently $f = 10,150$ lbs. per square inch. But in consideration of the remoteness of the possibility of all the wind load and all the brake load occuring together and at the same time as such a large impact load, which the braking load either removes or replaces, the allowed unit for the full combination of live, dead, impact, wind and brake loads is increased above that already determined by 25 per cent. There are thus two cases to be considered in every post—first, the ordinary combination at the ordinary unit; and, second, the extraordinary combination at the increased unit. In the present case, as with most cases of high towers and long spans, the second case governs and so the unit of 12,688 is used, necessitating 87 sq. ins. of section.

The feet of the posts are shod with a steel shoe plate 2 ins. thick which rests upon a steel bed plate 1½ ins. thick, the latter being anchored to the pedestals. The anchor bolts for bent 8 were 1¾ ins. diameter and 6 ft. long, two to each post, whilst to bent 7 were given 2½-in. diameter bolts 13 ft. 3 ins. long, two to each post. A few inches of each bolt project above masonry to receive nuts and washers above shoe plates, and the rest is built into the pedestals. A space is left around the bolt to allow of slight adjustment and then after the towers are completed this space is filled with cement grout.

One post of each tower was rigidly fixed to its bed plate, but the other three were allowed freedom to move under expansion or contraction from temperature or loading. Each of the posts adjacent to the fixed one is permitted to move in the direction of the line adjoining to the fixed one; that is, the one transversely to the viaduct axis, and the other longitudinally with the viaduct axis, while the diagonally opposite post can move in any direction. In other words, one shoe being fixed two had one degree of freedom and the other two degrees. Tongue and grooves in the bed and shoe plates, respectively, were used to accomplish this, and in the particular tower under notice the S. E. post was the fixed one. The tongues were 2½ x 5/16-in. flats and the grooves were planed out 7/16-in. depth.

Construction Details.—The viaduct was erected from the east end, and as the line itself was the only approach, the steel had to be forwarded from Montreal to the station on one of the existing railways nearest to the constructed track. This meant the carrying of all steel by the derrick cars from the east end storage to the actual site and over the erected portions. The process, therefore, was to erect bent 14 and place the 60-ft. girders from the east embankment. The derrick then moving into the 60-ft. girders erected bent 13 and the longitudinal bracing and the 30-ft. girders. This method, obvious and ordinary, was followed to bent 9. For the higher bents a horizontal temporary strut is employed to steady and support the first bent of a tower, being erected between it and the last completed tower. These temporary struts serve to stiffen the bent while the 60-ft. girders are placed and the derrick advanced to erect the rest of the tower. They were wooden in the case of the Boucanne River Viaduct, but are of steel for larger and higher towers.

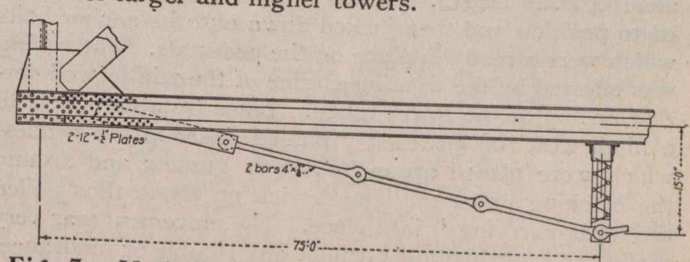


Fig. 7.—Method of Bolting and Trussing Bottom Chord for a One-Piece Erection.

When the tower 9-10 was completed, the next necessary undertaking was bent 8, which in its final position was to be 150 ft. west of bent 9. As this could not be erected from the west end, there being no line, it was essential to scheme out some method of building the complete tower 7-8 before the truss span which it was destined to carry. Although the river was but a foot or two in depth, falsework for the truss was considered too expensive and troublesome, and the plan now to be presented was selected as the most feasible and desirable means of erecting both the trusses and the tower. The outline of the scheme was the building of tower 7-8 on low staging close to 9-10 within reach of the derrick booms and approximately at its final level, and then rolling or sliding it westward into place. The company had before had experience with the moving of towers by this means and were satisfied from past successes that this was a possible and practicable scheme. Temporary wooden staging was built across the river upon which were laid the skidding or rolling track rails. These were laid at such a height that between their bases and the top of the pedestals of 8 there was just room for an ordinary railroad tie and an inch or two of blocking, Fig. 5. On top of these rails were then built the posts of bent 8. The base of bent 7 being 6 ft. higher than bent 8, this amount of timber blocking was built upon the rails 40 ft. west of bent 8 and upon this small timber tower bent 7 was erected. When the bracing and 40-ft. girders of this tower had been completed a stiff-leg derrick was built on top of the tower by the erection derrick on tower 9-10, Fig. 6.

Between the shoe plates of bent 7 and the blocking upon which they rested was placed another set of rails, so that when the tower had been rolled westward until blocking under bent 7, having passed pedestal 8, was about 8 ft. from pedestal 7 they would project over pedestal 7 and when staged or blocked, would form a track upon which

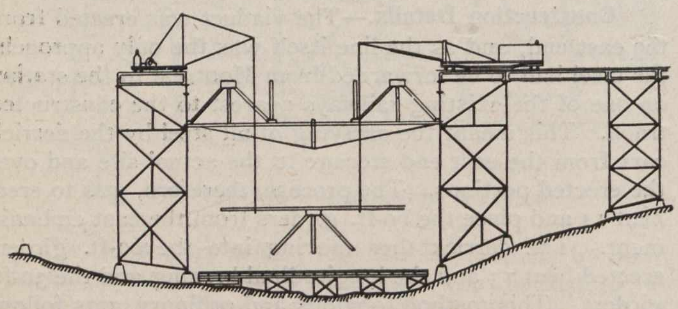


Fig. 8.—Method of Hoisting and Erecting Trusses.

the shoes of bent 7 could slide the last few feet of their travel, the shoes of bent 8, of course, still rolling. Thus for the first 135 ft. or so the two bents of the tower rolled on the lower tiers and afterwards the higher shoes slid on the upper tiers. The rollers were 2 ins. in diameter and about 4 ft. in length. In this manner the tower was rolled to position and then jacked down onto its anchor bolts which were already in place on the pedestals. The rolling was effected by the winding engine of the stiff-legged derrick on top of the tower rolled. Lines from its spools to a hitch west of pedestal 7 passed back to the tackles, which were placed around posts at gussets and around the blocking under 7. The derrick up above thus pulled the tower carrying it into place. The movement was very easily accomplished and without any trouble or hindrance.

The next problem was the erecting of the truss span. The two bottom chords were lowered piece by piece to the staging already referred to, a few feet only from the ground, and there they were bolted up and trussed as shown in Fig. 7, by temporary eye bars and short struts. By this means they were made strong enough to carry themselves and a part of their respective trusses as spans of 145 ft. 6 ins. length. The end vertical posts of the trusses had, of course, been already erected as part of the towers and the bottom chord and splices were arranged to permit of the trussed chord being hoisted vertically and immediately bolted to the short end sections, which were shop riveted to the gusset plates at the foot of the vertical posts incorporated into the towers. By this means the two bottom chords were hoisted into place and connected, the stiff-legged derrick on 7-8 taking hold of about 40-ft. out and the main traveller taking hold of about 50 ft. out. The bottom lateral bracing was then put in place and the chords thus stayed horizontally.

The next process (Fig. 7) was the erecting on the low staging of the web members of the two panels at each end of each truss; that is, the main compression diagonal, the first vertical and the main tension diagonal. These were then hoisted into place and immediately braced by the placing of the first floor-beam and sway-bracing. The second verticals being added, the remaining central portion of each truss was assembled down below with the four panels of top chord and then similarly hoisted up to position and bolted on. The intermediate floor-beams, sway-bracing and top lateral bracing followed and the addition of end sections of top chords and stringer girders completed the span. When sufficient riveting had been done the temporary eye bar trussing was removed and everything was ready for the laying of track and the passage of the erection derrick. The stiff-legged derrick was dismantled and the erection of the remaining towers was proceeded with on the same plan as above described for the eastern shore. The photograph of the placing of the last girder is interesting in that it shows the device whereby a brace frame is hoisted above the second girder of a span, so that immediately both girders are correctly placed they can be braced together.

The main erection derrick had an interesting feature in its telescopic booms. They were built of steel sections in two lengths each, and so arranged that the outer half could enter the inner half and the effective boom thus shortened. Holes and pins were provided in the webs of each position of each boom whereby various lengths could be obtained and retained. The capacity of the booms was 15 tons per boom when shortened down to about 40 ft., and 10 tons each when acting at their full extension of 62 ft.

The greatest weight lifted in this viaduct was about 20 tons, divided between the derrick and the stiff-leg erected on tower 7-8. This weight comprised the centre portion of the 150-ft. span, was 394,185 lbs., and the total in the viaduct 913 tons.

The steelwork was manufactured and erected by the Dominion Bridge Co., Limited, of Montreal, under the supervision of R. F. Uniacke, bridge engineer to the N.T.C.R. Mr. F. P. Shearwood is the designing engineer of the company, and Mr. J. Finley the erection superintendent, while the author as assistant designing engineer was in responsible charge of the designing of the work above described.

The year 1910 has stood the record year for the production of coal in British Columbia, with a total of 3,139,235 tons (2,240 lbs.). The production in 1912 amounted to 3,025,709 tons, which is the second highest annual production, labor troubles being responsible for the lesser output than that of 1910.

Engineers and contractors from many sections of the country are to gather at Cleveland, Ohio, September 17 and 18, on the occasion of the tenth annual meeting of the National Paving Brick Manufacturers' Association. In former years the association has held its annual meetings during winter months, but at the last yearly assemblage of the paving brick manufacturers, it was decided to hold future conventions during an "open season." This will afford, besides the usual programme of written papers, discussion and criticism of brick street and brick road construction methods while work on the highways is in actual progress. The large amount of construction work in the district, will afford splendid opportunity for investigation in a most practical way.

COAST TO COAST.

Saskatoon, Sask.—The city council has decided to prospect systematically for natural gas throughout this district, and Mr. W. R. Martin, a leading expert from Medicine Hat, has been engaged for that purpose. Mr. Martin will endeavor to trace the gas strata to where it comes closest to this city. Borings will then be made, and, in the event of the discovery of gas, the practicability of piping it to the city will thereafter be considered. It will be remembered that Calgary pipes her gas from Bow Island, a distance of 185 miles.

Saskatoon, Sask.—The construction of the big Dominion Government interior elevator at Saskatoon will be commenced upon within the next few weeks. At the outset, its capacity will be three and one-half million bushels. The plans, however, permit of its extension to twenty million bushel capacity, as occasion may later require. When here recently, Grain Commissioner Jones stated that the Saskatoon elevator would be able to handle 150 cars of grain per diem, and when completed would be one of the largest elevators on the continent, exceeding the elevator at Fort William, and having a larger capacity than either of the two Canadian Northern Railway elevators at Port Arthur. The structure will be of reinforced concrete, and absolutely fireproof. Contracts will be let before September 1st. The site will be served by the three railways. The location of the elevator at this city, of course, involves the establishment of a sample market also, and, in due course, a grain exchange.

Guelph, Ont.—At a recent meeting of the Sewerage and Public Works Commission, plans and specifications were presented for important improvements contemplated to the Neeve Street bridge. This shows that the board fully realizes that in order to prevent floods, such as occurred in the spring of 1912 and 1913 action must be taken this season while the water in the river is low. The plans call for another 24-foot arch on the north side of the bridge, and for other extensive improvements to the river and banks. It is proposed to clean out the bed of the river, both below and above the Neeve Street bridge for some distance, and also to erect a retaining wall for about 75 yards above the bridge, and also on the opposite side of the river below the bridge. Every effort will be made to allow plenty of room for the flood water to flow down the river without its flooding its banks. It is understood that this action on the part of the city will not in any way prejudice the case now before the courts, and the Board feel the necessity of taking immediate action to prevent a flood in 1914.

Regina, Sask.—Mr. A. S. Porter, who has been working diligently with a view to materially reducing the cost of production of power in this city, has offered to sell power to the city at $18 per horse-power per 10-hour day. The present rate for power is 1.1 cents per k.w., and Mr. Porter's offer is to sell to the city at .7 cents. It is not thought likely that the city will accept, however, as when the new power-house is completed, it is expected that the cost of production will be materially reduced.

Toronto, Ont.—Hon. Dr. Reaume is likely to introduce legislation at an early session changing the rural main road allowance from 66 feet, as at present, to accord with the newer standard. It is pointed out by the Government's engineers that the standard rural main roads are but 24 feet, and with allowance for paths of four feet on each side and ditches, 40 feet would be ample. The present road allowance leaves a vast acreage of land which might be planted, and forms many miles of growth by vicious weeds. The new standard road likely to be authorized on recommendation of the Highways Commission will be of slight grade 24 feet wide, and with very sharp curves obliterated. The data obtained by the department show that the use of motor cars in Europe has multiplied the traffic in the main highways ten times, with the consequent cost of maintenance in proportion.

Toronto, Ont.—The Harbor Commissioners are congratulating themselves on an advantage obtained by the building of the new western channel, which did not enter into their considerations at the time the new channel was planned. The good result obtained is the practical clearance from the bay of debris. Some time ago the suggestion was made by a prominent engineer that the bay water could be kept clear by the construction of a channel at a point about half way between Centre Island and Hanlan's Point, but owing to the big expense involved nothing was done in the matter. The result now obtained has been caused by a strong current which flows from the eastern channel west and out through the western channel and keeps the bay clear. A current strong enough for this purpose was not to be had when the old western channel was used, which was too narrow, and was only about 10 feet deep in places. The new wide channel is more than 18 feet in depth, and is apparently situated just right to catch the current.

Vancouver, B.C.—The fourth annual convention of the Pacific Highway Association met in Vancouver a week ago. Delegates from Seattle, Portland, San Francisco, Los Angeles and a number of other cities are in attendance. The association aims to secure the construction of a first-class road along the Pacific slope from as far north as possible. Except for two short breaks in British Columbia, the highway now stretches continuously from Hazelton, B.C., to Yuma, Ariz. During the winter large sections of the road are impassable, and the association is working to secure the co-operation of the different communities toward an early improvement of the conditions.

Ottawa, Ont.—It is understood that the Canadian Northern Railway is seeking from the Government running rights over the Intercolonial Railway, to take effect from the time when their Montreal to Port Arthur line is finished. The privilege will, of course, not be required until the construction of the line is much further advanced than it is at present. When it is completed next year, the Canadian Northern will have a line as far as Quebec, but will lack connection with the eastern seaboard. No definite plans have ever been formulated by the company to build a new line to St. John or Halifax, and apparently it has been intended to fill that gap by the acquisition of running rights over the Intercolonial. For that reason the application made to the late Government is again being pressed. The intention of the Government towards the application has not been officially stated, though it is known in connection with the question of Government assistance to the line that the matter has been under consideration from time to time. When the former ministry granted liberal bond guarantees to the Canadian Northern it was understood that if at any time the Intercolonial Railway desired running rights west over the Canadian Northern such would be granted. The same privilege is still available, but it is not known whether the Government will utilize it.

Stratford, Ont.—The city's sewage disposal plant, which has been in operation for only a year, is reported to be giving unsatisfactory service. F. A. Dallyn, C.E., of the Provincial Board of Health, recommends the following improvements to increase the efficiency of the system: New sedimentation beds and improved sprinkling equipment, including an auxiliary gas engine; a chlorination device, and a humus bed between the sprays and the creek into which the purified effluent empties.

Regina, Sask.—Mr. Moreley J. Donaldson, vice-president of the Grand Trunk Pacific Railway, while in the city recently stated that a contract would be let within the near future for the construction of the balance of the Regina-Moose Jaw line of his company. With regard to the boundary line, Mr. Donaldson stated that the laying of steel to the boundary would be completed within the course of a few days, and that the line would be ballasted to the boundary by September 15th sufficiently to permit operation. He also stated that the Great Northern Railway line would be completed to the boundary to connect with the Grand Trunk line as soon as the Grand Trunk Pacific line is completed.

PERSONAL.

GUSTAVE KAHN, general manager of the Trust Concrete Steel Company, of Canada, Limited, Walkerville, Ont., is at present in the West on a business trip.

HENRY F. CLAYTON, general manager of Boving & Company, of Canada, Limited, is spending the month in Western Canada on a tour in the interests of business.

L. S. BRUNER, manager of publicity for the Canada Cement Company, Montreal, is returning from a visit to the company's mills in British Columbia and the Western States.

R. F. UNIACKE, chief engineer of the Transcontinental Railway, has just returned to Ottawa from an inspection as far west as Winnipeg of the bridges and general condition of his road.

W. C. HARVEY is acting city engineer of Lethbridge, Alta., since the resignation of A. D. C. Blanchard. Mr. Blanchard is now associated with C. H. Rust in the city engineer's department of Victoria, B.C.

ARTHUR H. BLANCHARD, M. Can. Soc. C.E., professor in charge of the graduate course in highway engineering at Columbia University, has been retained by the State highway department of Pennsylvania as consulting engineer on the appraisal of certain toll roads.

CHARLES H. EDMONDS, for nine years works chemist and carbonizing engineer for the Birmingham Gas Department, and who subsequently erected and managed a large chemical plant at Middlesborough, England, for Messrs. Reckett & Sons, is now in this country.

J. G. McMILLAN, B.A.Sc., has returned from James Bay, where he has spent five months for the Provincial Government towards the establishment of a suitable harbor location for the proposed terminus of the T. and N.O. Railway. The work of sounding the Bay has been completed.

WILLIS CHIPMAN, of the firm of Chipman & Power, Toronto and Winnipeg, has been engaged by the city of Edmonton in the capacity of consulting engineer for a period of two years at an annual salary of $3,600, Mr. Chipman to visit the city four times in the year, the visits to aggregate thirty days spent in the city.

L. S. ODELL, B.A.Sc., a member of the firm of Linn, Collis & Company, Nipigon, Ont., was in the city last week completing arrangements and securing equipment for the commencement of their contract with the Canadian Northern Railway. This contract covers the concrete work in the construction of fourteen bridges on the eastern division of the Sudbury-Port Arthur line.

D. P. WAGNER and FRED. G. McALISTER have been appointed secretaries of the new Provincial Highway Commission, the former in the capacity of general secretary, and the latter as secretary in charge of statistical work. Both are graduates of the University of Toronto, and Mr. Wagner is a graduate of Oxford as well. Mr. McAlister has acted as secretary of the Good Roads Association for the past year.

JAMES NEWTON GUNN, general manager of the Studebaker Corporation, of South Bend, Ind., has resigned in order to meet the demands made upon his time and services by other clients. Mr. Gunn will remain upon the board of directors of the Studebaker Corporation, but will devote practically his entire time to the firm of Gunn, Richards & Company, engineers and accountants, of New York and Montreal With the assistance of the large staff of engineers and accountants employed by the latter firm, Mr. Gunn and his partner, Mr. Richards, are managers of, or engineers for, a large number of industrial and engineering concerns, several of which are in Canada. Gunn, Richards & Company have a Canadian office at Montreal under the management of H. Victor Brayley.

JOINT MEETING OF PEAT SOCIETIES.

The seventh annual meeting of the American Peat Society will be held jointly with the Canadian Peat Society at the Canadian Society of Civil Engineers' building in Montreal, August 18th, 19th and 20th. Papers to be presented and discussions will embody the results of progress in the production of peat fuel, peat gas and peat moss litter. Emphasis will be given the subject of peat and bog utilization in agriculture, both on this continent and in Europe.

COMING MEETINGS.

ONTARIO MUNICIPAL ASSOCIATION.—Annual Meeting to be held in Toronto, August 28th and 29th. Secretary-treasurer, Mr. K. W. McKay, County Clerk, St. Thomas, Ont.

THE NEW ENGLAND WATERWORKS ASSOCIATION.—Annual Convention to be held in Philadelphia, Pa., September 10th, 11th and 12th, 1913. Secretary, William Kent, Narragansett Pier, R.I.

THE ROYAL ARCHITECTURAL INSTITUTE OF CANADA.—Sixth General Annual Assembly will be held at Calgary, Alberta, September 15th and 16th. President, J. H. G. Russell, Winnipeg, Man.; Hon. Secretary, Alcide Chaussé, 5 Beaver Hall Square, Montreal, Que.

CANADIAN PUBLIC HEALTH ASSOCIATION.—Third Annual Meeting in Regina, September 18th, 19th and 20th. General Secretary, Major Lorne Drum, Ottawa; Local Secretary, R. H. Murray, C.E., Regina.

AMERICAN ROAD CONGRESS.—Annual Session will be held in Detroit, Michigan, from September 29th to October 4th. Secretary, J. E. Pennybacker, Colorado Building, Washington, D.C.

AMERICAN SOCIETY OF MUNICIPAL IMPROVEMENTS.—Twentieth Annual Meeting to be held in Wilmington, Del., October 7th to 10th. Secretary, A. Prescott Folwell, 15 Union Square, New York.

UNITED STATES GOOD ROADS ASSOCIATION.—Convention will be held at St. Louis, Mo., November 10th to 15th. Secretary, J. A. Rountree, L021 Brown-Marx Building, Birmingham, Ala.

AMERICAN ROAD BUILDERS' ASSOCIATION.—Tenth Annual Convention to be held in First Regiment Armory Building, Philadelphia, Pa., December 9th to 12th. Secretary, E. L. Powers, 150 Nassau Street, New York, N.Y.

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