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PETERBOROUGH WATERWORKS

DETAILS OF MODERN WATERWORKS PLANT—DESCRIPTION OF PUMPING EQUIPMENT

By R. O. WYNNE-ROBERTS, M.Can.Soc.C.E.

PETERBOROUGH, a city of about 22,000 inhabitants, derives its supply of water from the River Otonabee, about 2½ miles upstream from, and at a slightly higher elevation than, the centre of the city. The water is pumped direct to the consumers, and as the highest part supplied is only 100 feet above the pumps there is no difficulty in maintaining sufficient pressure.

A reinforced concrete dam, about 300 feet long, thrown across the river, impounds the water in the river for the development of power and as storage for the supply to the citizens. The depth of water behind the dam is about 12 feet and affords an available head on the

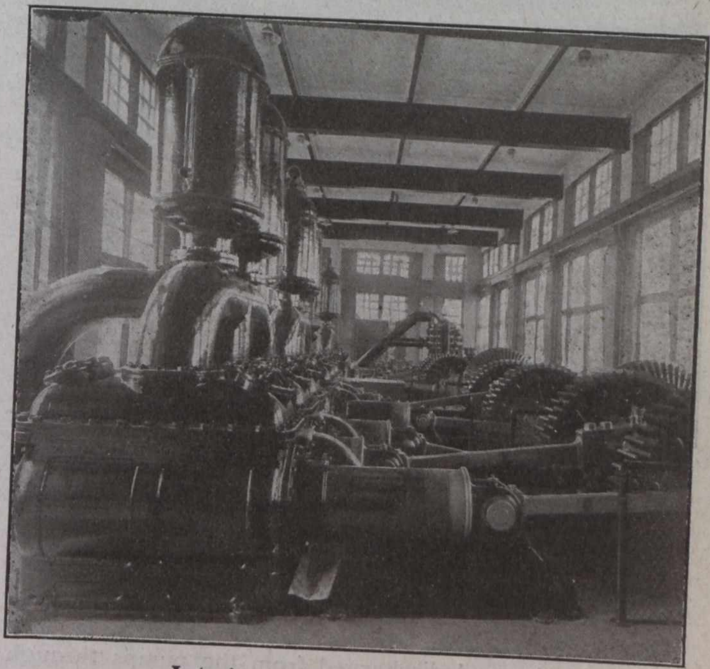
Reinforced Concrete Dam Across Otonabee River.

water turbines of an average of about 10 feet, and the suction lift of the pumps ranges from 3 to 10 feet. Provision is made to draw water from the tailrace, when frazil ice causes trouble above the dam, or when the stream is low and it is desirable to conserve the water for power, the suction lift in this case is about 18 feet.

The pumping station is a substantial building of reinforced concrete framework and brick panels and measures about 125 feet long by 40 feet wide. It houses three sets of water-driven horizontal single-acting triplex pumps, a Worthington centrifugal pump and an auxiliary steam pump. The two oldest pumps were installed in 1893 and are massively built. They have three plungers each 18 inches in diameter, 30-inch stroke, and are operated at 20 r.p.m. when run at full capacity. Each of these are capable of delivering 2¼ million gallons per day. The plunger heads are connected to the crank-shafts carrying massive cog-wheels which are driven by pinions from a horizontal shaft that transmits the power from the water turbines through bevel wheels. The water power is developed by vertical Samson turbines—62-inch wheels. Speed, 66 r.p.m.

The other triplex pump of similar construction was built in 1909 and has a capacity of three million gallons per day. The plungers are 21 inches in diameter, 30-inch stroke, and driven in a like manner to the other by a Samson vertical turbine, having a 68-inch wheel running at about 80 r.p.m. The bevel gear reduces the speed of the pumps to 22 strokes per minute.

There is a 3-stage Worthington vertical centrifugal pump, also driven by a Samson water turbine. The water-



Interior of Pumping Station.

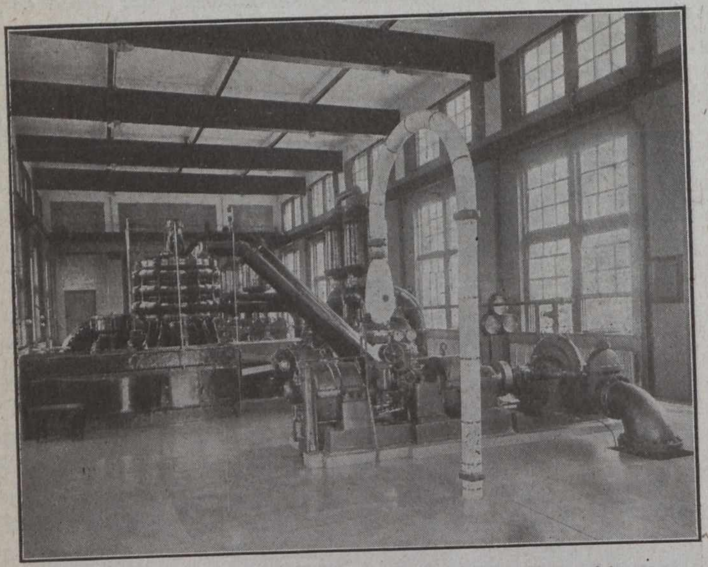
wheel rotates about 80 r.p.m. and by means of an increasing gear the pump is capable of being driven at about 640 r.p.m. The delivery from the pump is rated at about three million gallons daily.

The steam auxiliary pump, installed in 1915, is a De Laval single-stage turbine driven by a 400 h.p. steam turbine running at about 6,000 r.p.m. under 125 lbs. steam pressure. The pump is driven at about 1,385 r.p.m. by the steam turbine by means of the usual reduction gear. This pumping unit is split horizontally for easy access to all parts. The suction is 14 inches diameter and the delivery 12 inches. This pump is arranged to draw water either from above the dam with a suction lift of 3 feet or thereabouts or from the tailrace with a lift of about 18 feet, and is rated at six million gallons per day under a total head of about 228 feet. The condensing plant consists of a Schutte-Koerting multi-jet condenser and a De Laval centrifugal pump working under 35 feet head at 2,200 r.p.m. and discharging about 700 gallons per minute, and driven by a steam turbine of type T.A.

The pump-house is equipped with an overhead travelling crane carrying a hand-operated 10-ton block tackle.

The water-driven triplex pumps and water turbines were made by William Hamilton Co., Peterborough. The Worthington turbine pump was built by John McDougal Caledonian Iron Works Co., Montreal, and the De Laval steam turbine and pumps were supplied by the Turbine Equipment Co., Toronto.

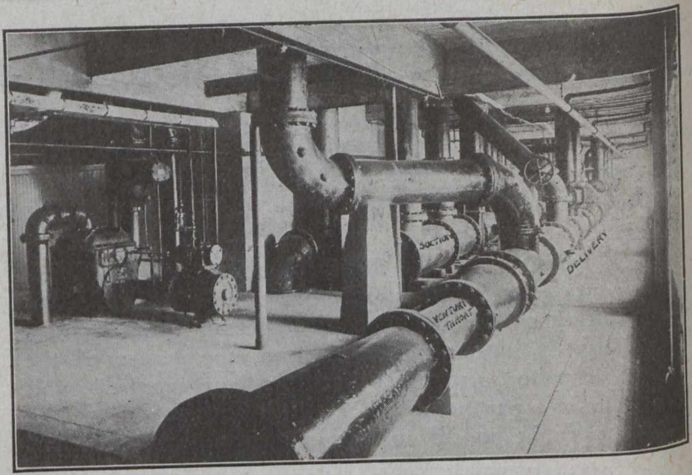
There are two 100-h.p. horizontal return tube boilers each 66 inches in diameter and 16 feet long, fitted with Cyclone shaking grates of 34 square feet in area. A fan to induce draft is belt-driven by a 6-h.p. vertical steam engine, and a 6-h.p. Foos gasoline engine is installed to operate the fan when the boiler furnaces are being started. The boilers are not heated until necessity arises. A Cochrane feed water heater and feed pump are installed. The boilers are placed in a separate adjoining building,



Another View of Pump-house, Showing Worthington Vertical Pump and De Laval Auxiliary Pump.

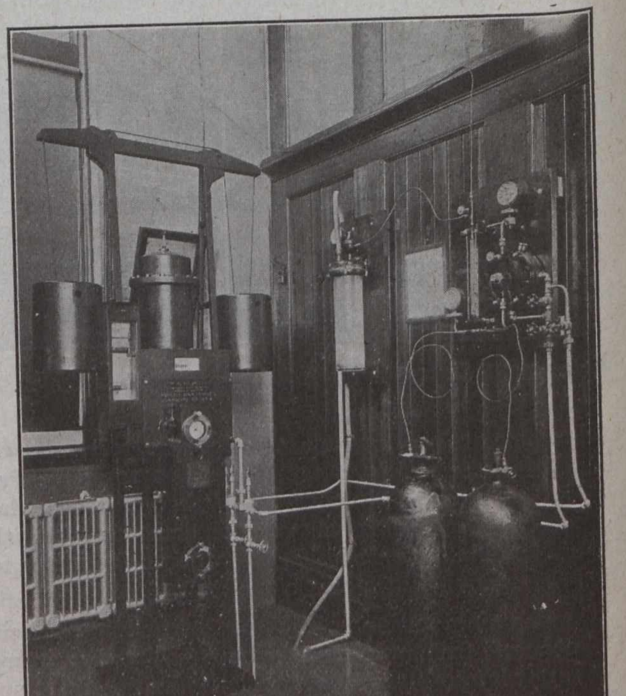
the base of which is reinforced concrete with a brick superstructure, and a laminated wood roof covered with 5-ply Barrett roofing.

The water is pumped from the river through a 30-inch suction header and discharged from the pumps through individual connections controlled by valves into a 24-inch delivery main, in which is inserted a 10-inch Venturi throat to measure the quantity pumped. The Venturi meter recording apparatus is in the office on the main floor, together with a self-recording Bristol pressure gauge and Wallace & Tiernan's chlorinating plant. The meter is type M Builders' Iron Foundry Co. make and needs no description, but it is interesting to note that the Venturi throat referred to not only measures the volume of water pumped but also controls the operation of the chlorinating plant. The differential pressures in the inlet and throat of the Venturi meter are transmitted to the Venturi meter recorder and also to the diaphragm which controls the admission of liquid chlorine according to the varying quantity of water that is supplied. The liquid chlorine is supplied in steel cylinders under considerable pressure and when it is decided what quantity of it must be used to sterilize the water it can be done easily by means of a glass gauge set against a graduated scale. Having noted the volume of water that is being registered by the meter and adjusting the chlorinator scale to suit the required proportion of chlorine to be added, the differential pressures at the Venturi throat automatically regulate the chlorinator for other rates of flow. The liquid chlorine meets with a small quantity of water in a glass vessel and the resulting



Showing Venturi Meter.

mixture is conveyed by a rubber pipe to the suction connection in the pipe gallery. The maximum quantity of liquid chlorine used, is stated to be about 3 lbs. per million gallons but the average in 1916 was 1½ lbs. The chlorine



View Showing Chlorinating Plant.

costs about 25 cents per pound delivered at the pump-house inclusive of all charges.

The pump-house, etc., are steam heated by means of a 30-h.p. horizontal return-tube boiler.

The force mains to the city consist of one 16-inch and one 18-inch cast-iron pipe. These continue well within the city boundaries. The normal pressure at the pumps is about 70 lbs. and the fire pressure from 100 to 130 lbs. per square inch. There are about 44 miles of mains ranging from 3 inches to 18 inches in diameter, all cast-iron pipe. There are also 4,323 services.

The total quantity pumped in 1916 was about 1,100,000,000 gallons, or an average of about three million gallons per day. The maximum peak rate was about 5½ million gallons per day. The average daily consumption of water in 1916 was about 136 gallons per capita. The cost of the water, inclusive of all charges, averaged about 4.95 cents per 1,000 gallons.

The Peterborough waterworks were first installed by a private company in 1882 and were acquired by the city in 1902.

The water commissioners are Mr. T. F. Matthews, chairman; Messrs. W. H. Moore and Robt. Hicks. Mr. R. L. Dobbin, B.A.Sc., is the waterworks superintendent, to whom credit must be given for the foregoing information, and Mr. Wm. Kennedy, Jr., Montreal, is the consulting engineer.

DESIGN FEATURES FOR CONCRETE AND REINFORCED CONCRETE CONSTRUCTION.*

THE span length for beams and slabs simply supported should be taken as the distance from centre to centre of supports, but need not be taken to exceed the clear span plus the depth of beam or slab. For continuous or restrained beams built monolithically into supports, the span length may be taken as the clear distance between faces of supports. Brackets should not be considered as reducing the clear span in the sense here intended, except that when brackets which make an angle of 45° or more with the axis of a restrained beam are built monolithically with the beam, the span may be measured from the section where the combined depth of beam and bracket is at least one-third more than the depth of the beam. Maximum negative moments are to be considered as existing at the end of the span as here defined.

When the depth of a restrained beam is greater at its ends than at mid-span and the slope of the bottom of the beam at its ends makes an angle of not more than 15° with the direction of the axis of the beam at mid-span, the span length may be measured from face to face of supports.

T-Beams.—In beam and slab construction an effective bond should be provided at the junction of the beam and slab. When the principal slab reinforcement is parallel to the beam, transverse reinforcement should be used, extending over the beam and well into the slab.

The slab may be considered an integral part of the beam, when adequate bond and shearing resistance between slab and width of beam is provided, but its effective width shall be determined by the following rules:—

(a) It shall not exceed one-fourth of the span length of the beam;

(b) Its overhanging width on either side of the web shall not exceed 6 times the thickness of the slab.

Beams in which the T-form is used only for the purpose of providing additional compression area of concrete should preferably have a width of flange not more than 3 times the width of the stem and a thickness of flange not less than one-third of the depth of the beam. Both in this form and in the beam and slab form the web stresses and the limitations in placing and spacing the longitudinal reinforcement will probably be controlling factors in design.

Floor-Slabs Supported Along Four Sides.—Floor-slabs having the supports extending along the four sides should be designed and reinforced as continuous over the supports. If the length of the slab exceeds 1½ times its width, the entire load should be carried by transverse reinforcement.

For uniformly distributed loads on square slabs, one-half the live and dead load may be used in the calculations of moment to be resisted in each direction. For oblong slabs, the length of which is not greater than 1½ times their width, the moment to be resisted by the transverse reinforcement may be found by using a proportion of the live and dead load equal to that given by the formula,

$r = \frac{l}{b} - 0.5$, where l = length and b = breadth of slab.

The longitudinal reinforcement should then be proportioned to carry the remainder of the load.

In placing the reinforcement in such slabs account may well be taken of the fact that the bending moment is greater near the centre of the slab than near the edges. For this purpose two-thirds of the previously calculated moments may be assumed as carried by the centre half of the slab and one-third by the outside quarters.

Loads carried to beams by slabs which are reinforced in two directions will not be uniformly distributed to the supporting beams, and the distribution will depend on the relative stiffness of the slab and the supporting beams. The distribution which may be expected ordinarily is a variation of the load in the beam in accordance with the ordinates of a parabola, having its vertex at the middle of the span. For any given design, the probable distribution should be ascertained and the moments in the beam calculated accordingly.

Continuous Beams and Slabs.—In computing the positive and negative moments in beams and slabs continuous over several supports, due to uniformly distributed loads, the following rules are recommended:—

(a) For floor-slabs the bending moments at centre and at support should be taken at $\frac{wl^2}{12}$ for both dead and live loads, where w represents the load per linear unit and l the span length.

(b) For beams, the bending moment at centre and at support for interior spans should be taken at $\frac{wl^2}{12}$ and for end spans it should be taken at $\frac{wl^2}{10}$ for centre and interior support, for both dead and live loads.

(c) In the case of beams and slabs continuous for two spans only, with their ends restrained, the bending moments both at the central support and near the middle of the span should be taken as $\frac{wl^2}{10}$.

(d) At the ends of continuous beams, the amount of negative moment which will be developed in the beam will depend on the condition of restraint or fixedness, and this will depend on the form of construction used. In the ordinary cases a moment of $\frac{wl^2}{16}$ may be taken; for small

*From the final report of the Special Committee on Concrete and Reinforced Concrete submitted January 17 at the annual meeting of the American Society of Civil Engineers.

beams running into heavy columns this should be increased, but not to exceed $\frac{wl^2}{12}$.

Flat Slab.—The recommendations in the following paragraphs relate to flat slabs extending over several rows of panels in each direction. Necessarily the treatment is more or less empirical. The coefficients and moments given relate to uniformly distributed loads.

Column Capital.—For computation purposes, the diameter of the column capital will be considered to be measured where its vertical thickness is at least 1½ ins., provided the slope of the capital below this point nowhere makes an angle with the vertical of more than 45°. In case a cap is placed above the column capital, the part of this cap within a cone made by extending the lines of the column capital upward at the slope of 45° to the bottom of the slab or dropped panel may be considered as part of the column capital in determining the diameter for design purposes. Without attempting to limit the size of the column capital for special cases, it is recommended that the diameter of the column capital (or its dimension parallel to the edge of the panel) generally be made not less than one-fifth of the dimension of the panel from centre to centre of adjacent columns. A diameter equal to 0.225 of the panel length has been used quite widely and acceptably.

Dropped Panel.—Generally, it is recommended that the width of the dropped panel be at least 4/10 of the corresponding side of the panel as measured from centre to centre of columns, and that the offset in thickness be not more than 5/10 of the thickness of the slab outside the dropped panel.

Slab Thickness.—The following formulas for minimum thicknesses are recommended as general rules of design when the diameter of the column capital is not less than 1/5 of the dimensions of the panel from centre to centre of adjacent columns, the larger dimension being used in the case of oblong panels. For notations, let

t = total thickness of slab, in inches;
L = panel length, in feet;
w = sum of live load and dead load, in pounds per square foot.

Then, for a slab without dropped panels,

$$\text{minimum } t = 0.024\, L\sqrt{w} + 1\tfrac{1}{2};$$

for a slab with dropped panels,

$$\text{minimum } t = 0.02\, L\sqrt{w} + 1;$$

for a dropped panel whose width is 4/10 of the panel length,

$$\text{minimum } t = 0.03\, L\sqrt{w} + 1\tfrac{1}{2}.$$

In no case should the slab thickness be made less than 6 ins., nor should the thickness of a floor-slab be made less than 1/32 of the panel length, nor the thickness of a roof-slab less than 1/40 of the panel length.

Bending and Resisting Moments in Slabs.—Analysis shows that, for a uniformly distributed load, and round columns, and square panels, the numerical sum of the positive moment and the negative moment at a vertical section of a slab taken across a panel along a line midway between columns, and another section taken along an edge of the panel parallel to the first section, but skirting the part of the periphery of the column capitals at the two corners of the panels, is given quite closely by the equation

$$M_x = \frac{1}{8}\, wl\left(l - \frac{2}{3}c\right)^2.$$

In this formula and in those which follow relating to oblong panels,

w = sum of the live and dead loads per unit of area;
l = side of a square panel measured from centre to centre of columns;
l_1 = one side of the oblong panel measured from centre to centre of columns;
l_2 = other side of oblong panel measured in the same way;
c = diameter of the column capital;
M_x = numerical sum of positive moment and negative moment in one direction;
M_y = numerical sum of positive moment and negative moment in the other direction.

For oblong panels, the equation for the numerical sum of the positive moment and the negative moment at the two sections named becomes

$$M_x = \frac{1}{8}\, wl_2\left(l_1 - \frac{2}{3}c\right)^2$$

$$M_y = \frac{1}{8}\, wl_1\left(l_2 - \frac{2}{3}c\right)^2$$

where M_x is the numerical sum of the positive moment and the negative moment for the sections parallel to the dimension, l_2, and M_y is the numerical sum of the positive moment and the negative moment for the sections parallel to the dimension, l_1.

What proportion of the total resistance exists as positive moment and what as negative moment is not readily determined. The amount of the positive moment and that of the negative moment may be expected to vary somewhat with the design of the slab. It seems proper, however, to make the division of total resisting moment in the ratio of ⅜ for the positive moment to ⅝ for the negative moment.

With reference to variations in stress along the sections, it is evident from conditions of flexure that the resisting moment is not distributed uniformly along either the section of positive moment or that of negative moment. As the law of the distribution is not known definitely, it will be necessary to make an empirical apportionment along the sections; and it will be considered sufficiently accurate generally to divide the sections into two parts and to use an average value over each part of the panel section.

Positive Moment.—For a square interior panel, it is recommended that the positive moment for a section in the middle of a panel extending across its width be taken as

$$\frac{1}{25}\, wl\left(l - \frac{2}{3}c\right)^2.$$

Of this moment, at least 25 per cent. should be provided for in the inner section; in the two outer sections of the panel at least 55 per cent. of the specified moment should be provided for in slabs not having dropped panels, and at least 60 per cent. in slabs having dropped panels, except that in calculations to determine necessary thickness of slab away from the dropped panel at least 70 per cent. of the positive moment should be considered as acting in the two outer sections.

Negative Moment.—For a square interior panel, it is recommended that the negative moment for a section which follows a panel edge from column capital to column capital and which includes the quarter peripheries of the edges of the two column capitals (the section altogether forming the projected width of the panel) be taken as

$$\frac{1}{15} wl \left(l - \frac{2}{3} c \right)^2.$$

Of this negative moment, at least 20 per cent. should be provided for in the mid-section and at least 65 per cent. in the two column-head sections of the panel, except that in slabs having dropped panels at least 80 of the specified negative moment should be provided for in the two column-head sections of the panel.

Moments for Oblong Panels.—When the length of a panel does not exceed the breadth by more than 5 per cent. computation may be made on the basis of a square panel with sides equal to the mean of the length and the breadth.

When the long side of an interior oblong panel exceeds the short side by more than one-twentieth and by not more than one-third of the short side, it is recommended that the positive moment be taken as

$$\frac{1}{25} wl_2 \left(l_1 - \frac{2}{3} c \right)^2$$

on a section parallel to the dimension, l_2; and

$$\frac{1}{25} wl_1 \left(l_2 - \frac{2}{3} c \right)^2$$

on a section parallel to the dimension, l_1; and that the negative moment be taken as

$$\frac{1}{15} wl_2 \left(l_1 - \frac{2}{3} c \right)^2$$

on a section at the edge of the panel corresponding to the dimension, l_2, and

$$\frac{1}{15} wl_1 \left(l_2 - \frac{2}{3} c \right)^2$$

at a section in the other direction. The limitations of the apportionment of moment between inner section and outer section and between mid-section and column-head sections may be the same as for square panels.

Wall Panels.—The coefficient of negative moment at the first row of columns away from the wall should be increased 20 per cent. over that required for interior panels, and likewise the coefficient of positive moment at the section half way to the wall should be increased by 20 per cent. If girders are not provided along the wall, or the slab does not project as a cantilever beyond the column line, the reinforcement parallel to the wall for the negative moment in the column-head section and for the positive moment in the outer section should be increased by 20 per cent. If the wall is carried by the slab, this concentrated load should be provided for in the design of the slab. The coefficient of negative moments at the wall to take bending in the direction perpendicular to the wall line may be determined by the conditions of restraint and fixedness as found from the relative stiffness of columns and slab, but in no case should it be taken as less than one-half of that for interior panels.

Reinforcement.—For a column-head section, reinforcing bars parallel to the straight portion of the section do not contribute to the negative resisting moment for the column-head section in question. In the case of four-way reinforcement, the sectional area of the diagonal bars multiplied by the sine of the angle between the diagonal of the panel and the straight portion of the section under consideration may be taken to act as reinforcement in a rectangular direction.

Arrangement of Reinforcement.—All bars in rectangular bands or diagonal bands should extend on each side of a section of maximum moment, either positive or negative, to points at least 20 diameters beyond the point of inflection, as defined herein, or be hooked or anchored at the point of inflection. In addition to this provision, bars in diagonal bands used as reinforcement for negative moment should extend on each side of a line drawn through the column centre at right angles to the direction of the band at least a distance equal to 35/100 of the panel length, and bars in diagonal bands used as reinforcement for positive moment should extend on each side of a diagonal through the centre of the panel at least a distance equal to 35/100 of the panel length; and no splice by lapping should be permitted at or near regions of maximum stress, except as just described. Continuity of reinforcing bars is considered to have advantages, and it is recommended that not more than one-third of the reinforcing bars in any direction be made of a length less than the distance centre to centre of columns in that direction. Continuous bars should not all be bent up at the same point of their length, but the zone in which this bending occurs should extend on each side of the assumed point of inflection, and should cover a width of at least 1/15 of the panel length. Mere draping of the bars should not be permitted. In four-way reinforcement, the position of the bars in both diagonal and rectangular directions may be considered in determining whether the width of zone of bending is sufficient.

Reinforcement at Construction Joints.—It is recommended that at construction joints extra reinforcing bars equal in section to 20 per cent. of the amount necessary to meet the requirements for moments at the section where the joint is made be added to the reinforcement, these bars to extend not less than 50 diameters beyond the joint on each side.

Provision for Diagonal Tension and Shear.—In calculations for the shearing stress which is to be used as the means of measuring the resistance to diagonal tension stress, it is recommended that the total vertical shear on two column-head sections constituting a width equal to one-half the lateral dimension of the panel, for use in the formula for determining critical shearing stresses, be considered to be one-fourth of the total dead and live loads on a panel for a slab of uniform thickness, and to be three-tenths of the sum of the dead and live loads on a panel for a slab with dropped panels.

The calculation of what is commonly called punching shear may be made on the assumption of a uniform distribution over the section of the slab around the periphery of the column capital and also of a uniform distribution over the section of the slab around the periphery of the dropped panel, using in each case an amount of vertical shear greater by 25 per cent. than the total vertical shear on the section under consideration.

Bending Moments in Columns.—Provision should be made in both wall columns and interior columns for the bending moment which will be developed by unequally loaded panels, eccentric loading or uneven spacing of columns. The amount of moment to be taken by a column will depend upon the relative stiffness of columns and slab, and computations may be made by rational methods, such as the principle of least work, or of slope and deflection. Generally, the larger part of the unequalized negative moment will be transmitted to the columns, and the column should be designed to resist this bending moment. Especial attention should be given to wall columns and corner columns.

The world's largest artificial harbor is planned for Rotterdam. It will be 28 feet deep and cover 766 acres.

PNEUMATIC CAISSON WORK ON THE PETITCODIAC RIVER BRIDGE PIERS*

DESCRIPTION OF AN UNUSUAL PIECE OF FOUNDATION WORK—VELOCITY OF RIVER, RAPID RISE OF TIDE MADE CONSTRUCTION DIFFICULT—METHODS OF CONSTRUCTION

By E. M. ARCHIBALD, A.M.Can.Soc.C.E.

THE Petitcodiac River is situated at the northern extremity of the Bay of Fundy, and in this neighborhood there occurs the highest tides in the world, the government records showing a rise of 45 ft. at spring tides. At the city of Moncton, located on the Petitcodiac River, some twenty miles from its mouth and at the head of navigation, the rise of tide is only thirty feet, owing to the higher level of the river at that point. Here, at each incoming tide, appears the "bore," a tidal wave due to the onrushing water crowding into the ever-narrowing channels of the river. The bore, although appearing at other points on the Bay of Fundy, is at its highest at Moncton and is most readily seen from that point. This bore varies in height from a few inches to at least four feet—many claim six feet—depending on the stage of tide and velocity of the wind.

Historical.—Directly opposite Moncton is a populous farming country and after long, unsuccessful endeavors to have a highway bridge built by the government, these farmers resolutely and at great sacrifice, raised enough money to start work on the piers for a bridge and managed to keep the project going until after two years' work a bridge was in existence—this was in the year 1869.

Old timers tell of the difficulties in placing the wooden cribs forming the piers; how one after another broke loose from the moorings and was swept away by the swift current, and how eventually the piers were completed, wooden spans constructed on shore and floated out on scows into place—altogether an undertaking of no mean engineering ability. Somewhere about this period, the Provincial Government were brought to a realization of their responsibility and bought the bridge.

Unfortunately, its benefits were not long enjoyed, for in the autumn of 1869 what is known as the Saxby Gale occurred, causing great havoc to shipping and to property, and in the tremendous tide that ensued, the superstructure was washed away and the piers scoured so much that they settled from eight to ten feet in a few cases. However, reconstruction was proceeded with, the piers added to and the superstructure rebuilt. After about twenty years' service the superstructure was again renewed. At the present time the existing superstructure is approximately twenty-five years old, and has been in urgent need of renewal for several years. The life of the old piers, now nearly fifty years old, is past.

About ten years ago natural gas was discovered at Hillsboro and in order to reach the nearest market in Moncton, the mains had to cross the Petitcodiac River. A 10-inch gas main was added to the loading of the old bridge, carried by projecting brackets. Several attempts have been made to lay a gas main in the bed of the river, but without any success. The old bridge continues to be the weak link in the transportation of this very necessary feature to the domestic and commercial life of Moncton.

Location of New Bridge.—The location of the new bridge is 33 ft. up river from the centre line of the old bridge. In considering the location for a complete new bridge, the provincial engineer, Mr. A. R. Wetmore, decided to keep as close as possible to the old bridge for two reasons: (1) Financial—in that the old structure would be of very great service in the erection of the new bridge; (2) soundings showed a ridge of rock closer to the surface of the water at this location than for some considerable distance either up or down river. On the other hand, it was very necessary not to undermine the existing foundations of the old piers. The adoption of the pneumatic system of sinking caissons was decided upon as the least dangerous to the old piers.

River Conditions.—Working conditions in the Petitcodiac River are particularly severe. There is the bore already referred to, the high tides, the high velocity of the current, but perhaps the most discouraging feature is the muddy condition of the water due to the tide sweeping over the extensive mud flats, picking up the mud or silt, carrying it in suspension and depositing it in eddies and at high and low-water intervals. The velocity of the current reaches 10 miles an hour at half tide on what are known as spring tides occurring for a week at each full and new moon periods. At neap tides occurring during the first and last quarters, the tide gradually falls to as low as 20 ft. rise and the current diminishes correspondingly. This rise of tide occurs in a period of three hours, then there is a lull of twenty minutes at high-water before the tide falls perceptibly, after which it recedes very rapidly for three and a half hours, then more slowly. There remains a period of about four hours at slack low water for subaqueous work before the arrival of the next succeeding tide.

Another very annoying condition was the rapid shifting of the channel below the bridge site, caused at times by very high winds and at other times by a freshet or heavy run-off of fresh water carrying tremendous quantities of mud in suspension from the upper reaches of the river and deposited on the flats, causing a bar to build up and thereby prevent the water from running off. A rise in low-tide elevation of as much as 42 inches was observed from this cause.

Ordinary soundings showed from six inches to eight feet of silt on the bottom, below which the material was so firm and compact that an ordinary sounding bar could not penetrate it.

Construction Equipment.—In considering the method to be adopted for construction of the substructures, the weakness of the old bridge and piers and the fear that it would be swept entirely away by the severe ice conditions existing in the winter and early spring, together with the large amount of traffic passing over it, led to the decision to use a cable-way spanning the river, and derricks at each pier for handling all material. It was estimated that the largest block of granite required in any pier would not exceed four tons and that this would probably be the maximum load to be carried on the cableway, but to provide for emergencies, it was designed for a maximum load of six tons. To allow a sufficient factor of safety for the long span of 1,600 ft. between towers, a 2-inch carrying

*Paper read before the Canadian Society of Civil Engineers, February 22nd, 1917.

cable was advisable, but as the contractors had in stock a 1¾-inch cable 1,620 ft. long, it was decided to use it and reduce the working strain in the cable by building higher towers and allowing a heavier sag in the cable. The cable, as installed, measures 1,591 ft. between towers and can be adjusted for a minimum sag of sixty feet. The cable in stock was short by about 200 ft. of the length required to span the river and over the towers to the anchorages. This was overcome by splicing short pieces to each end, the connection being made by drop-forged sockets. The anchorages were made by excavating to a depth of 6 feet in the ground 20 feet wide and 20 feet long, building a crib platform carrying 100 tons of stone for ballast, the main cable being carried to the bottom and around a very large birch log. A turnbuckle at one end provides for taking up about 6 feet of slack in the cable. The tower on the Moncton side is 90 feet high and has a base of 23 feet wide by 30 feet lengthways of the cable. The front legs of the tower are set on a batter of one in five, which falls within the resultant of the forces acting on the tower. The tower on the Albert County side of the river is on a higher elevation and in consequence was only built 85 feet high. The centre line of the cable was located 14 feet up river from the centre line of the new bridge. The stringing of the main cable across the river presented a problem in itself as it was necessary to keep the cable out of the swiftly running current of the river, which, with the maximum sag permissible and with the weight of this main cable, required the strength of the supporting wire to be 14 tons during erection. This was solved by using the two ⅝-inch wire ropes, later used for the hauling cable, as a carrier for supporting the weight of the main cable, this main cable having snatch blocks wired to it every 200 feet, which travelled along on the two supporting wires. The cableway outfit is a Flory design, the engine having two 8½ x 12-inch cylinders, link motion, and two 42-inch drums. The hoisting and hauling ropes are each ⅝-inch diameter wire rope.

For mixing concrete, a battery of two ½-yard Smith mixers was used, located directly under the cableway. Gravel was shovelled into ½-yard cars at the storage pile, cement added to make the required batch and the whole hauled up an incline to the mixer platform.

Two 40-h.p. locomotive boilers were used for the first year's operations, previous to the installation of the pneumatic plant. Two hundred and twenty additional boiler horse-power was added later to take care of the pneumatic work.

The compressor plant was located immediately alongside the boiler plant and consisted of one 1,150 cu. ft. Ingersoll-Rand compressor, 18 x 24¼ x 24 and a second of 1,250 cu. ft. capacity with cylinders of 24 x 24¼ x 30. The air, after being compressed, passed into a 42-in. x 14-ft. receiver, thence by two lines of 4-in. pipe located along the old bridge, to the work.

A derrick car was used in the granite storage yard for unloading the granite from incoming cars and afterwards reloading on small cars running to a point directly under the cableway, whence it could be picked up and carried to the pier under construction.

The floating plant consisted of one "A" frame derrick scow 60 x 25; one 68 x 28 and three 45 x 16, all with flush decks. This floating plant was seldom used on account of difficulty in handling in this turbulent river.

Water obtained from the city water supply was carried through 1,200 ft. of 2-in. pipe to the boilers and mixers and a 1-in. pipe line extension to the different piers under construction.

Construction Work.—The north abutment was excavated about 6 ft. below the surface of the ground, after which piles were driven and the concrete poured directly from the mixers. This abutment was above high tide.

Pier "B" was foundationed on piling 10 ft. above low-water mark, the excavation being sheet piling. The only difficulty encountered here was due to the heavy sediment deposited by each tide, which had to be scoured off before concreting could be continued.

The excavation for the south abutment was carried to rock, which was reached 12 ft. above ordinary low-water mark; the excavation was sheet piled to prevent the banks falling in when the tide filled the excavation. A concrete footing course 6 ft. thick was poured and the granite then started. No difficulties were encountered with this pier.

No. 4 pier was founded on rock at extreme low-water mark, the rock being levelled off at slack-water periods to a level bottom 6 inches below extreme low tide. A grillage 30 inches thick was accurately placed on this levelled-off bottom, forming a footing for the granite pier. No great difficulties were met with in the erection of this pier, except for the precautions to prevent the tide washing away the green concrete hearting. Tarpaulins were used for this purpose, covering the entire work and being roped down solidly to the birch grillage just before the arrival of the bore. Notwithstanding all precautions taken, a small quantity of the concrete hearting would be washed away and a deposit of mud left by the tide, which would have to be scoured off before pouring any more concrete. To give some idea of the force of the current at this pier, it may be mentioned that granite blocks placed on top of the tarpaulin to hold it down, were displaced by the current, and in one instance several granite blocks containing as much as a yard apiece were washed off the pier.

When taking soundings at the location for No. 3 pier, it was found that the bottom was scoured clean to the rock and was only 5 feet below low-water for over half the length of the pier at its centre, but dipped to a depth of 9 feet at both upstream and downstream ends of the pier. It had been the intention to use the pneumatic system in foundationing this pier, but as a result of these soundings it was decided to build an open caisson with the sides fitting the bottom and with a puddle chamber on the outside, so that the water could be pumped out and the rock cleaned before concreting. This caisson was built on shore with a puddle chamber 2½ feet wide surrounding it. Just previous to high tide it was launched, towed to its position and moored through the ebb tide. Towards the end of the ebb tide it was placed accurately in position and ballasted with rails. The puddle, consisting of clay and gravel, was placed in the puddle chamber and completely covered with gravel in bags to prevent scour by the tide. Just when ready to start pumping operations a period of heavy rains came on, bringing down with the fresh water large quantities of silt, which deposited on the bars in the river and caused a rise of 3½ feet in low tide elevation, and this rise of water caused endless trouble, since the puddle chamber had not been designed for so much pressure and when pumped out it would blow repeatedly. Fortunately, the caisson had been bulkheaded into four water-tight sections. Pumps were placed in one section at a time at the early part of low-water period and by pumping each compartment separately the bottom was cleaned of sediment and concreting carried on at low-tide intervals. Several tides were required to fill each compartment on account of the delays caused by cleaning operations at each tide. A tarpaulin was battened down on top of the green concrete at the completion of each tide's operations to prevent scour.

Concrete was stopped 2 feet below low-tide mark, after which 3 feet of hardwood timber grillage was used to form a footing for the granite pier.

The work already described was performed from the time operations started in July, 1915, to the end of the same year.

Pneumatic Caissons.—The responsibility for the design of the caissons was entirely in the hands of the contractors, except for the outside dimensions. Further, the engineer's requirements that no concrete be deposited through water, in addition to the launching and holding problems, presented a set of conditions none too easy of solution. To fulfil the first requirement, the caisson was divided into three parts with two water-tight bulkheads, so that any one compartment could be pumped out and concrete deposited in the dry space during low-tide periods. This shortened the time of pumping and permitted longer time for concreting and also reduced the buoyancy, as only sufficient weight was necessary to overcome the displacement of the pumped-out chamber and the natural buoyancy of the caisson.

No. 1 caisson, as launched, was built 18 ft. 9 ins. high, with 18 ins. of concrete on the roof of the air chamber, floated 13 ft. 6 ins. deep and weighed 370 tons. At the time of launching, three-quarters of an hour before high tide, there was only 15 ft. of water available at the end of the launchways. The intention was to launch early enough before high water to permit the caisson to be towed to place and the moorings secured before the turn of the tide, then to place enough timber on the top of the caisson so that when sunk in place the top of the caisson would be above low water. Intentions are not always carried out and it proved so in this case, for the launching was not successful. The caisson slid off the ways, sideways, just after starting. However, after jacking up, the above programme was followed. The caisson was launched successfully, towed across the river, moored, and several courses of timber placed. The moorings held through one ebb and one flood tide, but parted at about half ebb the second day. It is only fair to state that this accident was due to an extremely heavy surge of the current. The caisson grounded on a mud flat about six hundred feet below the bridge and partially rolled over, enough to expose one cutting edge at low tide. Here she lay, the sand scouring until she had enough bearing to carry, but resting in a hole in the sand 18 feet deep and sand banked up to the water's edge within 30 feet of the caisson. How, after rolling over several times, she was finally floated and brought back to place, might be interesting, but the writer can assure you it was just a succession of difficulties, not the least of which was the continued rainy weather. Simply, it consisted of righting the caisson and holding until she could be pumped out and nursed into a convenient place for handling.

(To be continued.)

TRANSCONTINENTAL OF AUSTRALIA.

Up to July 29th, 1916, the cost of the east-west transcontinental line of Australia from Kalgoorlie to Port Augusta was $20,683,970, exclusive of rolling stock and stores on hand. Rolling stock had cost $3,352,361. Recent advices say that 917 miles have been laid and that only 41 miles are required to complete the railway.

A NEW ORGANIZATION TO HELP WIN THE WAR.

The engineers and technical men belonging to the various branches of the profession in Ontario have recently organized to lend their technical knowledge in the service of their country.

The new movement is known as the Joint Committee of Technical Organizations, Ontario Branch, and is governed by a strong committee composed of representatives from all the known engineering associations of the province.

Representatives have been selected from such institutions as the Canadian Mining Institute (Toronto branch), Canadian Society of Civil Engineers (Toronto branch), Ontario Association of Land Surveyors, Society of Chemical Industry, Canadian Section (Ontario members), Engineering Alumni Association of the University of Toronto, Engineers' Club, Toronto, Royal Canadian Institute, Canadian Engineers (Military District No. 2), American Society of Mechanical Engineers (Ontario section), American Institute of Electrical Engineers (Toronto section), Institute of Electrical Engineers (Ontario members), Ontario Association of Architects.

As the members of the committee are probably as busy men as are to be found in the community, meetings are held once a week only, but a small executive meet three or four times weekly and report at the weekly meeting to the representatives of the above-mentioned societies.

That there is ample scope for good work from such a galaxy of knowledge as is represented at these meetings is evidenced from the fact that after less than a month's existence matters in hand embody nearly all interests connected with the war, from aiding recruiting in the engineering branches of the service, to giving technical advice when requested to the Munitions Board, munition manufacturers, and others engaged in war work.

The committee is making a canvass of the province to register all engineering ability of every sort and description, and cards have been distributed to the members of all the societies, asking for accurate information as to each man's special experience and qualifications.

This information should prove useful when obtained, as Ontario is called upon to organize her resources, mental as well as physical, to defeat the common enemy.

Through the efforts of this committee similar organizations are being brought together in the other provinces, and already Nova Scotia, Manitoba, Saskatchewan, British Columbia and Alberta have active work started to organize engineers for any work that may be delegated to them.

The objects of the association are clearly defined in their rules, which read as follows:—

"The object of the Joint Committee shall be to devise ways and means by which engineers and other technical men may, as a result of their special training and experience, render assistance in the development and government of our Dominion. The immediate aim shall be to evolve a plan or plans whereby such of these men as for business, family, or other reasons, are unable to go to the front, may be used for war purposes at home in such a manner as their special technical training may make them most valuable."

Any engineers not affiliated with any of the above associations and willing to lend a hand should apply to the secretary's office for a registration card.

The office of the Joint Committee is in Room 710 of the Excelsior Life Building, Toronto Street, Toronto.

CONSTRUCTION METHODS FOR ROGERS PASS TUNNEL.*

By A. C. Dennis, M.Am.Soc.C.E.

THE Rogers Pass Tunnel is a double-track tunnel, slightly more than five miles long, being part of a local improvement, about 10 miles long, of the Canadian Pacific Railway Company's main line through the Selkirk Range, in British Columbia. The map and profile are shown on Fig. 1.

The writer, knowing that the time of construction of this tunnel would weigh heavily with the railway company in considering bids, gave considerable thought to devising some quick method of construction. The usual American method of driving a top heading and taking out the bench with a power shovel was obviously too slow, and the ventilation would be too difficult, for a tunnel of this length. The European method, of a bottom heading and stoping out the rest of the section, would be too expensive, under existing labor costs. Shafts or adits were impracticable. It was finally concluded that a working tunnel or "pioneer heading" entirely outside of the regular tunnel section would be economically justified, under the existing conditions.

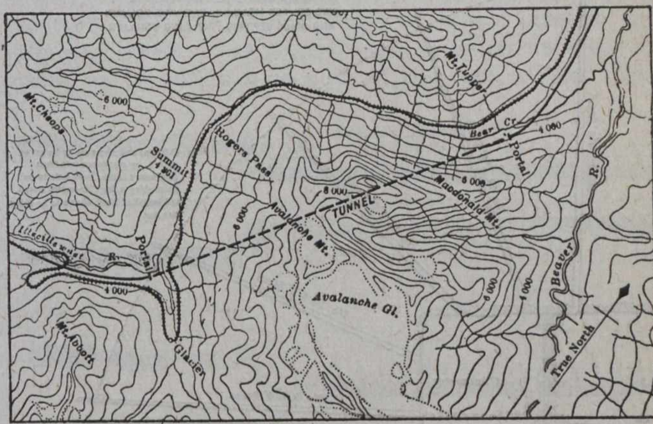
It was possible to keep the heading progress, enlargement drilling, enlargement blasting, and mucking going continuously, without any interference with one another whatever.

Bids were asked for this tunnel work early in 1913, and the contract was let to Foley Brothers, Welch and Stewart, railway contractors, in June, 1913. This firm was not the lowest bidder, but undertook to do the work more quickly than other firms, its estimate being based on the proposed pioneer method, Mr. Stewart, of the contracting firm, and John G. Sullivan, M.Am.Soc.C.E., chief engineer of the Canadian Pacific Railway, being willing to take the risk of this untried method. The first study contemplated a top heading, but, at Mr. Sullivan's suggestion, this was changed to a location midway in the full section, so that the heading could be made smaller and still permit holes to be drilled far enough to break to the full section; otherwise, the work has been done as planned by the writer, and under his superintendence.

Plant.—There was a power plant at each end, with five 150-h.p. boilers, equipped with special grates and induced draft, so that slack coal could be used for fuel. The boilers were run generally much beyond their nominal rating.

Each end had a capacity of about 4,000 cu. ft. of low-pressure air; that is, air at from 100 to 125 lbs. at the compressor, with from 90 to 100 lbs. at the drills, and a capacity of 1,500 cu. ft. of high-pressure air; that is, from 1,000 to 1,100 lbs. pressure at the compressors, and from 800 to 900 lbs. pressure at the charging stations. Ingersoll-Rand compressors were used, two-stage for low and four-stage for high pressure, with inter-coolers. The steam ends were compound condensing. The compressed air passed through an efficient after-cooler, so that little trouble was caused by the freezing of moisture in the compressed air when exhausted by drills, pumps, and blowers. The plant was properly put in and properly looked after, and caused practically no delay.

Wall-plate drifts were started in January and February, 1914, as soon as the approach cut was down to the wall-plate grade, and finished to rock in July. From these drifts a stope was started in March, and segment timbers were placed for about 100 ft. Owing to a change of plan by the railway company, this segment timber had to be removed and rebuilt to a larger section, so that stoping and timbering ahead of this point could not be resumed until June. The timbering from the wall-plate up was completed in October. A locomotive crane was found very useful for taking the timber and other material from the framing yard and delivering it in the wall-plate drifts. After the wall-plate drifts met the main heading, in July, the material from the break-ups or stopes was hauled to the first cross-cut and out of the pioneer heading, so that it would not interfere with the timbering. Previous to that time it was dumped at the portal and reloaded by the steam shovel, together with the material of the approach cut. The material from the break-ups was passed down through a chute into the small tunnel cars. Considerable turning and crushing of the wall-plate occurred, necessitating a great deal of propping of the segments.



PLAN AND PROFILE OF ROGERS PASS TUNNEL

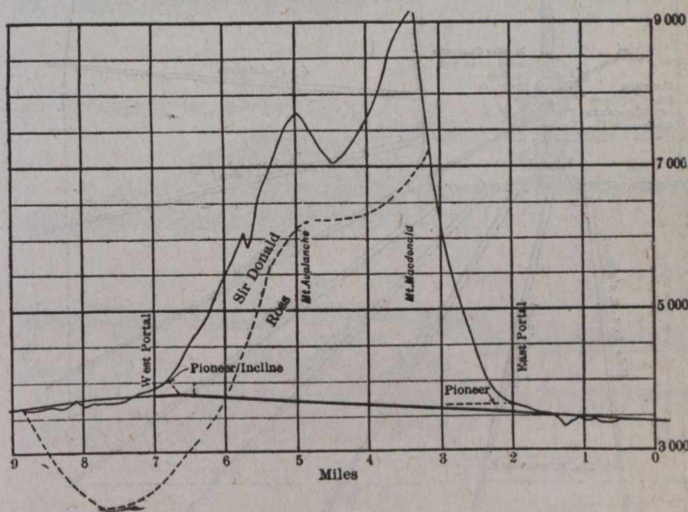


Fig. 1.

In August, the steam shovel, having finished the approach cut, started on the tunnel excavation, being operated by compressed air. During the day shift, the wall-plate was supported on temporary batter posts as the shovel progressed, and the night shift straightened the sides and set the plumb posts and lagging. During the night shift, the locomotive crane delivered the material required for the segment timbering the next day. After being impressed by the crushing of the wall-plate and segment timbers, it was with considerable anxiety that the undermining of the wall-plate with the power shovel was attempted. The characteristic of the ground, of re-

*Abstract of paper read before the American Society of Civil Engineers.

maining firm for a short time after being opened up, was encouraging. As it turned out, the fear of loosening part of the tunnel by the side sliding in, while the wall-plate was carried on batter posts, was groundless. There was no settlement of the wall-plate of more than 1 in., and the work was hastened and cheapened by this method much more than by driving drifts to place posts before excavating the core of the tunnel. The excavation of this earth section was finished in December, 1914.

Pioneer Headings in General.—The pioneer tunnel, in rock, was 7 ft. high and 8 ft. wide. It was driven with light hammer drills, using hollow steel, with water attachments. Three drills, in general, but four in the hardest rock, were used in a heading. Spare drill machines, for the replacement of drills out of order, were kept conveniently at hand in the heading. No repairs were made underground. The hammer drills are convenient and rapid, the delay and expense of their constant breakage perhaps balancing the advantage of speed under ordinary conditions. The drills are mounted on a light horizontal bar, about 18 ins. below the roof line. Air and water are taken over the muck pile, or on hooks in the side, by a single hose line for each, to a manifold from which short individual hose lines supply the drills.

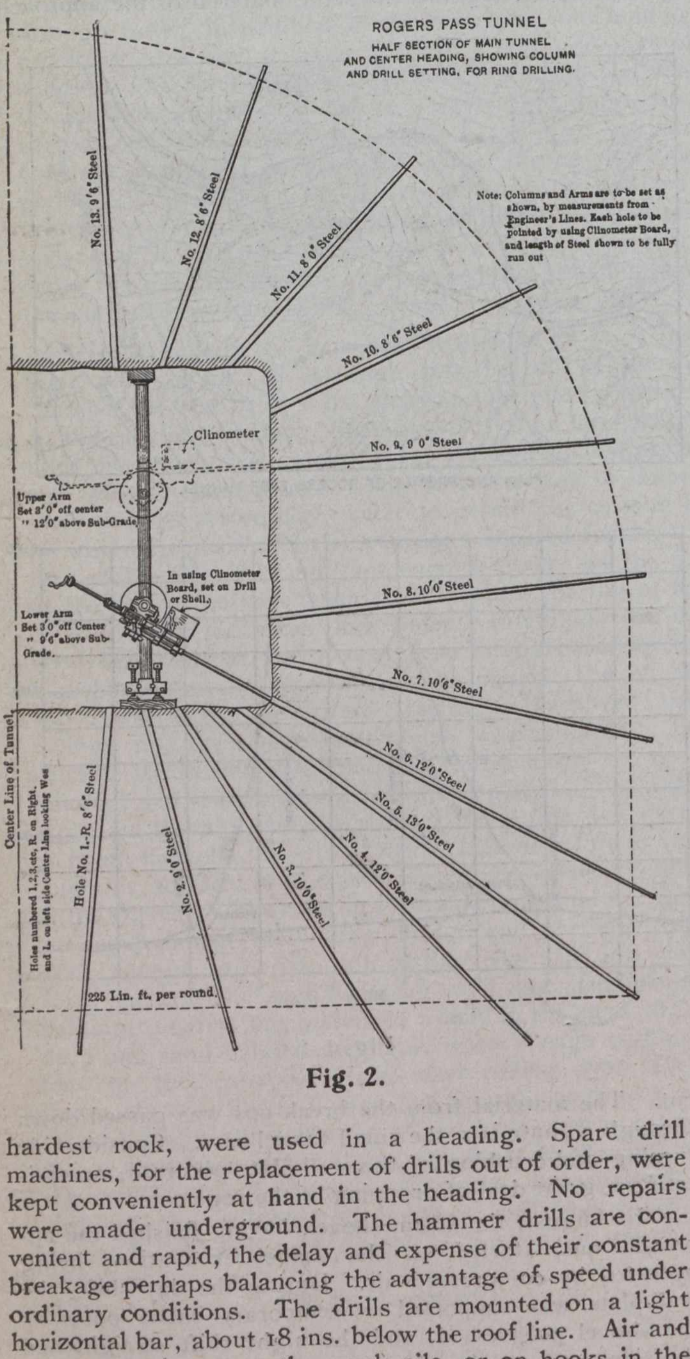


Fig. 2.

Light cars (½ cu. yd.) were used for muck, and the latter was taken off the track, instead of building sidings for this purpose. Shovelling plates were used at the face and on the side away from the track for some distance back of the face, in order to facilitate the handling of empty muck cars. The ventilating pipe was a 12-in. wooden water pipe connected to the Connersville blowers used for the exhaust. This pipe was hung on the side away from the track, close up to the roof and was carried to within 20 ft. of the face. Little damage was done to this pipe by blasting. The blowers were started exhausting when the first shot was fired, or a little before, and were run for 20 minutes. The men got back to work in from 5 to 10 minutes. No compressed air was allowed to be blown out for ventilating purposes. After a round was shot, the drillers followed the smoke back, barring down the roof, bringing explosives to re-shoot, and wetting down the muck pile, sides, roof, and face with water hose. The muckers cleared the track and began loading the muck which was scattered back.

When no further blasting was required, the lights were hung, the foreman sighted the line and grade point in the face, and the drilling gang set up the horizontal bar, placed their drills and proceeded. There was rarely any muck to be handled before the drilling could be started, as it was thrown back from the face by the heavy loading in the bottom holes and the fact that they were shot last, for this purpose. There were two helpers to three drills, and they brought up and changed the steel and adjusted the drill machines. When the drilling from the upper set-up was completed, the drillers took down the machines and carried them back, with the hose connections still attached, and oiled them up. After the mucking was done, the bar was dropped to the lower set-up, near the floor, and the drills were set to drill the bottom holes or lifters. The drills were carried forward, put on the bar, and were drilling sometimes in less than 2 minutes after the bar was dropped. While the bottom holes were being drilled, the muckers laid the track, adjusted and covered the mucking sheets with muck, and brought up the explosives. The holes were loaded by the machine men, helpers, and foremen.

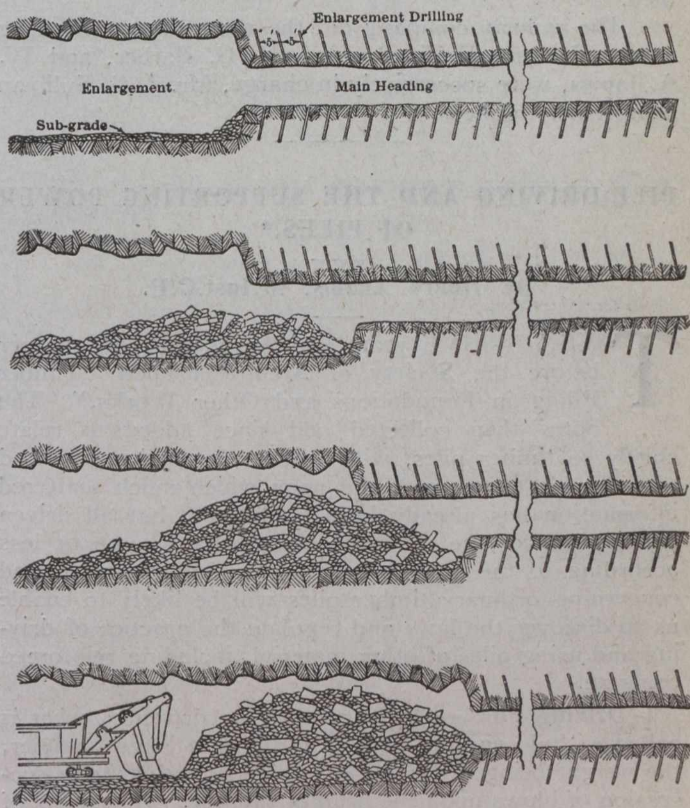
For the small part of the tunnel where re-shooting was not necessary, an 8-hour shift could do two rounds per shift, or a little better. Two men pick down the muck, and three men load the car and push it out, while three others stand by with an empty car, ready to put it on the track and load it. The three men taking out the loaded car return near the face with an empty car, take it off the track, and rest until the load comes out. The men get a rest from the monotony of steady continuous shovelling, and the empty car is available at once after the load goes back. The pipes for ventilating, and for air and water were laid by a pipe man and helper, who looked after several headings.

Doing this work with muckers was unsatisfactory. Muck cars were taken from the heading back to a siding by a single mule, and from there to the dump by a two or three-mule team driven tandem, until this method became inadequate, and then compressed-air locomotive haulage was substituted for the long haul. The heading muck cars, after the shovel and switching track had cleared a cross-cut, were taken to the cross-cut, pulled up an incline trestle by air hoist and cable, and dumped into standard-gauge cars. The cross-cuts are from 1,500 to 2,000 ft. apart. Air pressure was maintained at about 90 lbs. at the drills, which required 125 lbs. at the compressors toward the end of the work.

Main Heading.—The main heading, generally known on the work as the "Centre Heading," was entirely through the rock section. It was 11 ft. wide and 9 ft. high, the centre line being the same as that of the completed tunnel and the bottom being 6 ft. above the sub-grade. The position and size were such that lateral holes could be drilled from this heading to break the enlargement to the required dimensions. The air, water, and ventilating pipes for this heading were branches from the mains laid in the pioneer heading. Access to this heading was obtained through the cross-cuts from the pioneer, and muck was handled around the enlargement operations by the pioneer route. This heading was generally driven in a westward direction, on account of the drainage. The system of driving was similar to that in the pioneer. The rounds averaged about 7 ft., and 32 holes were drilled in the hardest rock. The main heading was sometimes driven from several faces. The average daily progress per heading at the east end was slightly more than 16 ft., and the maximum monthly progress was 621 ft. The average daily progress per heading at the west end was 20 ft.; the maximum monthly progress was 762 ft.

Enlargement.—Drilling: The enlargement drilling, after some experimenting, was done as shown by Fig. 2. Each hole was pointed by clinometer, the column carrying the drill being set always at the same distance off the centre line, and the arm for the lower and upper sets being always the same distance above the sub-grade. Line and levels were furnished by the railway company's engineers, and a string was stretched by which the columns and arms were located. Each drill hole had its proper distance from the arm. The drill holes were thus bottomed at a regular distance beyond the neat line of the completed excavation. The holes, being bottomed with reference to the line and grades given by the engineers, were not affected by irregularities in the heading driving. The columns were set by men for that purpose, so that the drillers and helpers had only to do the drilling. The drill steel was brought to the drillers, and the dull steel was taken away. The drillers and helpers were paid their wages in any event, but the footage for each man was kept, and if the price set per foot drilled amounted to more than his wages, he was given the difference as a bonus check. Air and water connections were made for every third ring of holes, and only one drill machine, though handled by each runner of the three daily shifts, completed the three rings, and then moved to the head of the line, taking the next three rings. Congestion of men and material was thus avoided, and each man had a fair chance to work on an equal quantity of hard and soft rock. There was extreme variation in the quantity drilled by different men and in different rock. The same man might do only 6 ft. a shift in the hardest quartzite, and more than 100 ft. per shift in the softer schist. New men, after a month's practice, generally made more footage than men of long experience in mining. In general, it was found better to train green men than to try to get men accustomed to piston drills to learn to run hammer drills. Most of the rings were 6 or 6½ ft. apart. When explosives rose in price it was found economical to space the rings 5 ft. apart, as the extra drilling cost was balanced by the saving in explosives, with the added advantage that the muck was broken into smaller pieces and scattered farther back. Where the roof was soft and full of slips, so that trouble was anticipated, the upper set of arms on the column was lowered 1 ft., in order to leave some trimming of the roof to be done by jackhammer, flat holes and light blasting. The air and water for the enlargement drilling, as well as the supplies, came by the pioneer tunnel and the cross-cuts, so that this drilling was not disturbed by the enlargement blasting. The drilling for the last mile, where no pioneer tunnel was driven, was started at the middle and progressed toward the portal, the track, pipe, etc., being removed as the drilling was finished. The stopping of the pioneer tunnel was well-timed, as the main heading was driven and the enlargement drilling completed just in time to avoid delaying the enlargement blasting and mucking at the east end.

Generally, from ten to fifteen rings were kept loaded ahead. Any part of a hole which had not broken, and could be found, was reloaded and shot with the next ring. Generally, a little muck was left in the face by the power shovel in order to prevent the first ring from scattering back too far. If the previously shot material had not broken to the required width, however, all the muck was loaded, and jack-hammers were used to drill up this tight rock, after which it was shot before the regular rings were blasted. Several bottom rings were first blasted, then a top and bottom ring were blasted together until the muck piled up to within 4 or 5 ft. of the roof. Then blasting



LONGITUDINAL SECTION, SHOWING ENLARGEMENT BLASTING

Fig. 3.

was discontinued, and the men scaled and trimmed the roof, working from the muck pile. (Fig. 3.) Where no holes had to be reloaded, rings could be blasted at intervals of from 15 to 20 minutes. The blasting was done with a battery in the main heading, and the bottom holes were all loaded ahead, the wires being wound up and stuck in the holes, from which they could readily be pulled out and connected. The upper holes were loaded, but no primers were put in until ready to blast. The holes were loaded to within 4 ft. of the collar, whether sprung or otherwise.

Concreting.—About 1½ miles of the tunnel, including the soft ground at each end, required concreting. This work was sublet to the Bates and Rogers Construction Company, of Chicago and Spokane. The concrete section is heavily reinforced. The sub-contractors used wooden forms, and deposited the concrete from a platform near the roof reached by an inclined trestle. The concrete mixer was on a car, and the materials were on other cars back of it. The concrete from the mixer flowed into a small car which was hauled by cable up the trestle incline to the high platform, from which it was shovelled into the forms. Much of the lining required back as well as front forms, and the space behind the back forms was filled with rock or wood. This back-form and back-filling work was slow and expensive, especially where there were only a few inches between the back forms and the rock. The concreting has been well done, but the work is behind time, owing largely to conditions which the sub-contractors could not control.

The tunnel has been finished 11 months ahead of the contract time, and for a substantial sum less than the price bid. There have been no strikes, or interruptions of the work. The management has had the necessary money and authority, and has been given a free hand, both by the railway company and the contractors, and has had the loyal support and valued assistance of the men.

The railway company did the necessary engineering work. Messrs. F. F. Busteed, H. G. Barber, and W. A. James, were successively in charge, Mr. J. G. Sullivan being chief engineer.

PILE-DRIVING AND THE SUPPORTING POWER OF PILES.*

By Henry Adams, M.Inst.C.E.

TWENTY-FIVE years ago the writer read a paper before the Society of Architects upon "Timber Piling in Foundations and Other Works." The notes then collected and since added to relate chiefly to timber piles, with which the writer has had considerable experience, and upon which much scattered information has already been published; but all driven piles, whatever the material, must behave more or less according to the same laws, and the knowledge gained concerning ordinary timber piles will be likely to enable us to discover the laws and regulate the practice of driving and using piles of other material, including reinforced concrete.

Driving Piles.—Ordinary piles are driven by what is called a "pile engine." It is virtually a large hammer, the weight being arranged to fall freely and strike a succession of blows upon the head of the pile. It consists of a tall framework, with vertical guides on the face to keep the hammer or ram in a direct line with the head of the pile. The base of the pile engine is placed just above the finished level of the pile head, so that in driving long piles a high framework is required. It is, however, sometimes impossible to get the framework high enough for this, and it is then set 6 ft. or 8 ft. above the finished level, and a punch, dolly, or follower, of hard wood, hooped at both ends, is used on the head of the pile when it gets as low as the base of the frame, but the blow is not so effective and the method should be avoided when possible; it is said to reduce the effect of the blow one-third, more or less, according to the rigidity of the material. The length of a pile is generally determined by the local conditions of site and soil, the sectional area chiefly upon the load it has to sustain, usually the ratio—

$$\frac{L}{A} = \tfrac{1}{4} \text{ to } \tfrac{1}{8}$$

but no general rule can be laid down, as it depends to some extent upon the unsupported length above ground.

*From a paper read at a meeting of the Concrete Institute held December 21st, 1916.

Pitching and Driving.—In pitching a pile care must be taken that it is started in the right place, as it cannot be shifted, but if the point is not truly in line with the axis of the pile, or gets pushed to one side by meeting an obstruction before it has entered very far, the lower end of the pile will be drawn over to the side to which the point leans. In spite of the greatest care they will sometimes be found slightly out of position, and they have then to be drawn back into place by chains, twisted like a surgeon's tourniquet, while being bolted to the other timbers. If it be necessary for them to be scarfed, the upper portion can be adjusted by cutting the scarf a little out of line to suit. All piles are not required to be vertical; in building jetties the outside row of piles is often doubled, the outer pile being a raking one, at 15 to 30 deg. from the vertical, for increasing the stability, acting like a buttress. At the corners of jetties the outer piles are usually raking both ways, say, about 15 deg. from the vertical. When necessity arises piles may be drawn from the bed of a tidal river by lashing empty barges to them and letting them lift by the tide. Against a river wall, or round the foundations of a bridge, piles should be sawn off by a diver as low as he can get at them in preference to drawing them, to avoid any risk of scour and undermining of the foundations taking place. On land a pile may be drawn by lashing a short piece to the top and then prising it up by another baulk used as a lever, or by a pair of powerful jacks.

Weight of Ram.—One of the most interesting questions in connection with pile-driving is the proportion between the weight of ram and the fall to produce a given result. The ram usually weighs from 5 cwt. to 30 cwt., and is allowed to fall, say, from 6 ft. to 20 ft. Upon a superficial consideration it would seem that a ram of 5 cwt. falling 20 ft. would produce the same result as a ram of 20 cwt. falling 5 ft., as they would both have 5 ft.-tons energy, but the proportion of the total energy ($W h$) which is usefully expended in sinking the pile depends *inter alia* upon the ratio of the weight of ram to the weight of the pile. Some of the total energy is always wasted.

A light ram with a long fall will not have the same effect as a heavy ram with a short fall. In practice it is found that with too great a fall the effect of the blow is to bruise and "broom" the head of the pile, or to shiver the timber instead of to force it downwards. A heavy ram, producing the same effect in distance driven as a light one with greater fall does less injury to the pile. Dobson says: "In working with a fall from 12 ft. to 20 ft. it is common for every tenth pile to be more or less shaken." Of course, he meant to say "one pile in ten." It is as if the top of the pile were driven down while the bottom remained stationary, owing to the inertia of the mass of the pile.

For the first few blows the pile goes down a considerable distance, which gradually becomes less at each blow until the resistance is so great that it will not go any further, or, as it is technically called, "refuses." Were the material of the pile perfectly rigid and inelastic, the impact, however slight, would produce an infinite pressure, but the material is very elastic, and so the fibres of the pile are compressed without the point going any further, and the amount of this compression and the elasticity are strikingly shown in the rebound of the ram when the pile refuses.

The Supporting Power of Piles.—The sustaining power of a pile depends chiefly upon three circumstances: (1) The resistance at the point or shoe to further penetration; (2) the friction of the earth on the sides of the pile; and (3) the strength, as a column, of the pile above the ground or above the firm subsoil.

Although it would seem that these are simply elements, each of which could be fairly estimated, the complexity of the case is shown by the numerous formulæ which give results ranging up to about 450 per cent.

In Newman's "Earthwork Slips and Subsidences" the frictional resistance of timber piles is stated to be less through wet soils than dry, in the following proportions:

In sandy gravel, 5 to 10 per cent. less.
In sand, about 12 per cent. less.
In sandy clay or gravelly clay, about 40 per cent. less.

Experiments with cast-iron piles by McAlpine gave about ½ ton per square foot of surface as the supporting power from friction when sunk 20 ft. to 30 ft. in rocky gravel. He considers it would amount to 3 tons per square foot in fine earth, but this seems to be an extravagant assumption. In experiments made previous to the sinking of concrete piles for the works of the Vienna-Danube Sand Dredging Company in 1909 it was found that the frictional resistance was about 14.19 lbs. per square inch of surface = 2,054.36 lbs. per square foot, or just over 18 cwt.

G. B. Bidder was of opinion that "in clay or wet soils it was not advisable to trust a greater weight than 12 tons upon each pile, but in gravel there was scarcely any limit to their vertical bearing strength."

French engineers (*vide* Berg's "Safe Building") allow a pile to carry 50,000 lbs., provided it does not sink perceptibly under a ram falling 4 ft. and weighing 1,350 lbs., or does not sink ½ in. under thirty blows.

A common rule for safe dead load on each pile is 5 tons per square foot of cross-section in soft ground, or 1 ton per inch side of square piles in firm ground.

The New York Building Regulations permit a load of 20 tons per pile, but the size is not specified.

The writer believes they use Wellington's formula for safe load.

Haswell (Minutes of "Proceedings" of the Institution of Civil Engineers, cxv., p. 318) says: "In deciding upon a factor of safety in a formula for pile-driving, the following elements must be considered: The friction of the machine; the resistance of the atmosphere to the fall of the ram and the cushioning on the head of the pile, however square it may be dressed off; the want of verticality both in the fall of the ram and in the plane of the pile, and the consequent lateral vibration; the inertia; the vibration and condition of the soil. Were all the conditions known definitely and allowed for, a factor of safety of 2 would be ample, but as the formulæ do not take account of all the conditions a larger margin is necessary. In some ascertained supporting powers recorded by Trautwine they were found to be from 2.3 to 3.7 times greater than given by the formulæ.

There seems to be no general rule as to the factor of safety it is desirable to adopt; the practice appears to vary from 2 to 10, the former being suitable for dead loads and the latter for live or vibrating loads. Intermediate factors would be produced with varying proportions between the dead and live loads. Obviously, unless the ultimate resistance given by the formula is reliable, the resulting factor is unknown.

In Dobson's "Foundations and Concrete Works" (Weale's Series) we are told that "of the comparative effect of impact and pressure in driving piles we as yet know nothing, and the question is so complicated, from the great number of points that have to be taken into consideration in reducing the results of experiment into a definite form from which some rule for our guidance might be obtained, that we can only lay down in general terms the following empirical rule that in ordinary cases if a pile will safely resist an impact of 1 ton, it will bear without yielding a pressure of 1½ tons," and he gives $W v$ = impact, therefore safe load = $1.5\ W v$.

Dobson is, however, wrong in his theory; he assumes that the force of a blow is measured simply by the product of the weight into the velocity, and this assumption leads him to conclude that a 1-ton ram with a fall of 16 ft. will have the same effect on the head of a pile as a 2-ton ram falling 4 ft., while the former takes double the expenditure of power to raise it. In other words, he says $f = m v$, instead of $f t = m v$, which is the well-known formula where f = force, t = time, m = mass = $\frac{w}{g}$ v = velocity —that is, a force f, acting for time t, will move a mass m, with a velocity v—but action and reaction are equal in magnitude but opposite in direction; therefore, a mass m moving with a velocity v, and expending its energy in time t, will produce a mean pressure—

$$f = \frac{m v}{t}.$$

Weight = product of mass into force of gravity, or $W = m g$, therefore

$$m = \frac{W}{g}, \text{ or } f = \frac{W v}{g t}, \text{ instead of } f = W v$$

as Dobson puts it. The same error is made by Molesworth, and a table of results is given assuming that the force of the blow varies as the square root of the fall instead of directly as the fall. The product $m v$ gives the momentum in its original sense. It is also known as quantity of motion, but it cannot be compared with force or pressure unless time be taken into account.

A worked example will perhaps make this clear—

Let $W = 0.25$ tons, $h = 20$ ft., S = set at head of pile = ¾ in., then

$$v = \sqrt{(2 g h)} = 35.7 \text{ ft. per second,}$$

but v varies from 35.7 to 0 while passing through the ¾ in., therefore mean $v = \frac{35.7}{2} = 17.85$ ft. per second, and ¾ in. = 1/16 ft.; therefore time occupied in passing through ¾ in. = 1/(17.85 × 16) = 1/285 of a second. Then by formula—

$$f = W v / g t = (0.25 \times 35.7/32 \quad 1/285) = 79.5 \text{ tons}$$

but if the exact fraction be taken it will give 80 tons mean pressure. By formula—

$$W v^2 / 2 g = W h = 0.25 \times 20 = 5 \text{ ft.-tons,}$$

or a mean pressure of $\frac{5}{1/16}$ = 80 tons, as before.

It will be observed that S is taken as the set of the head of the pile, and attention must be directed to one very important point. The distance the head of the pile moves after being struck is the criterion of the resulting force in pounds, but only part of this force is returnable as supporting power; some of it is expended in compressing the pile, and any formula that does not take account of the compression of the pile as well as the penetration of the point can only be approximately true. The resistance at the head of the pile begins at zero and terminates at such a pressure that the total movement multiplied by the average pressure equals the foot-pounds energy of the blow.

The test of a pile at Royal Victoria Dock, London, is recorded in "Engineering," December 29, 1899, p. 826, as follows:—

Pitch-pine pile, $12\frac{1}{2} \times 11\frac{1}{2} = 143.75$ sq. ins., 37 ft. long = 36.8 cu. ft., at, say, 50 lbs. per cu. ft. = 1,840 lbs. Weight of ram = 12 cwt. = 2,016 lbs.

Driven in peaty ground a distance of 36.1 ft., going $2\frac{1}{4}$ ins. to four 8-ft. blows when driving was stopped.

Loaded with rolled joists balanced on head.

Load 10	tons	No sinking	Remained thus	24 hours
" 20	"	1/16 in.	" "	48 "
" 30	"	1/16 in.	" "	0 "
" 37.5	"	3/16 in.	" "	7 days
" 56.9	"	¼ in.	" "	4 weeks

No further sinking. Load being removed, pile rose 3/16 in.

The rising of the head of pile when the load was removed showed the amount of temporary compression in the timber, and the missing 1/16 in. may have been further compression held back for a time by the friction of the earth, at the sides, and possibly reappearing some time later, a sort of *hysteresis*. The result of the test shows that the pile was practically safe with a load of 60 tons, but, like other tests that stop short of completion, it did not show that it would have failed with double the load or more.

The sort of test that is required to prove the accuracy of any formula is to measure the distance moved by the head and by the point during, say, the last four blows, and take the average for each. Cut off a foot from the top of the pile to receive the load on a good surface. Carefully level through from a fixed bench mark to find the level of the top of unloaded piles. Calculate the safe load on the pile and load it up until the head of the pile has sunk to the amount to which it was previously compressed (*i.e.*, previous average movement of head—average movement of point); note this load. Then continue loading, and note level of pile at each ton addition until the head of the pile has sunk, say, double the previous compression—*i.e.*, previous average movement of head, minus average movement of point. Then leave it loaded for twenty-four hours, and note whether it has sunk further and how much. Remove the load and note the result.

The difficulties in the way are: (*a*) Finding enough load; (*b*) supporting the load on the pile without friction; (*c*) measuring the results accurately; (*d*) avoiding personal danger throughout the test.

Reinforced Concrete Piles.—The advantages of reinforced concrete piles are so manifest that they need no express recommendation here. The chief physical differences from timber piles, as regards driving and their supporting power, are due to their extra weight and their friable nature. They should be made with slow-setting cement six weeks before driving, but if made with quick-setting cement they must be left eight weeks, because in the latter case the outside hardens first and leaves the interior soft. They should be driven by steam or drop hammer with a 3-ton ram, having a fall of 3 ft., with a steel helmet filled with sawdust, and preferably without a dolly.

In America the use of a water jet is found to greatly facilitate the sinking of concrete piles.

Hollow cylindrical reinforced concrete piles have been used at Southampton, Newcastle-on-Tyne, and Liverpool. They are lighter and cheaper than solid piles and more effective. Those at Brockelbank Dock, Liverpool, were 20-in. diameter. Reinforced concrete piles of circular section are easier to drive than square piles, and as they have no sharp angles are less liable to be damaged by coming into contact with boulders, etc.

As reinforced concrete piles are made horizontally, care must be taken in lifting them; the points of attachment for lifting should not be less than half the length apart, and if lifted with one end on the ground the attachment should be one-third the length from the other end.

The reinforced rods (about $2\frac{1}{2}$ per cent.) should preferably be hooked at the top and electrically welded together at the bottom. They should be bound helically by, say, ¼-in. wire, 4-in. pitch, carefully secured at the top.

REPORT OF COMMITTEE ON RECOMMENDED PRACTICE FOR STANDARDIZATION OF FILTERS.

Some years ago (May 12th, 1913) the American Society of Mechanical Engineers upon the suggestion of Mr. P. N. Engel appointed a committee to investigate and report on how to rate the capacity of mechanical filters.

This committee, of which Geo. W. Fuller, of New York, was chairman, has now presented its report and this, together with some discussion on it, is printed herewith:—

Your committee, appointed to make recommendations as to how to rate the capacity of mechanical filters, desires to submit the following report:—

Municipal vs. Industrial Field.—(2) The field of mechanical filtration may be arbitrarily yet definitely divided into two parts: One, the purification of drinking water or water for domestic supply, and the other the purification of water for other purposes, such as industrial uses.

Municipal Practice Substantially Uniform.—(3) On account of its importance and the large expenditure involved, especially in connection with municipal plants, much time and study have been given to all features of the filtration of water for domestic use. A large amount of data gathered through laboratory tests, and experience covering long periods in the practical operation of municipal plants, have brought into quite uniform adoption by all engineers engaged in such work the use of a rate of filtration of 2 gal. per min. per sq. ft. of filtering area for domestic supply.

Departures from Normal Municipal Rate Permissible.—(4) While stating as a matter of information that such a rate is applicable in the great majority of such cases, your committee does not feel warranted in setting forth this rate as one to be adopted for all cases. As a matter of fact, the installation of a municipal filtration plant usually is done, and always should be done, under the advice and supervision of a competent filtration engineer engaged for the purpose, and the rate of filtration as well as other points of construction and operation should be left to his judgment, based upon the local conditions that may exist. For this reason, your committee feels that it is advisable and in accordance with the spirit and intent of your instructions to refer herein chiefly to the filtration of water for other than municipal purposes.

Various Views Canvassed.—(5) Your committee in considering this subject has sought information and assistance from many sources, and we desire herewith to express our appreciation of the many courtesies extended to us by those thus called upon for data or comment.

Gravity vs. Pressure Filters.—(6) It may be well here, in view of the misunderstanding that seems to exist to some extent, to make some reference to the two different types of mechanical filters known as the gravity type and the pressure type. With both types, purification is accomplished by passing the water through a filtering bed, which in practically all cases is sand, and the purification is dependent upon the property or power of the bed of sand to remove suspended impurities from the water passing through. This property is one inherent in the filter bed itself, and, while it will be affected by the rate at which the water passes, it is not altered by the incidental fact of the water being or not being under more than atmospheric pressure. While, with additional pressure available, more water can be forced through a pressure filter than a gravity filter of given size, there is no difference in the principles or methods employed that warrants a higher rate of filtration with such filters than is acceptable for open or gravity filters when similar results are to be obtained.

Experience Leading to Standardization.—(7) From the information gathered it was made apparent that experience has already brought about a substantial unanimity of opinion and practice on the part of all those who, as engineers, chemists or manufacturers, are brought into close contact with the field of mechanical filtration as to the limits within which permissible rates of filtration must fall. A very definite rate has become established in connection with municipal work, and indeed if there had been anything like the publicity in connection with filtration of water for other purposes that has obtained in connection with gravity filters such as are installed for municipal work, it is probable that there would have been no occasion for such investigation and report as this committee has been called upon to make.

Tendency to Over-rating.—(8) While our investigation has made the above situation apparent, it has also developed the fact that there have been many filters installed in which the rate of filtration per unit of area is beyond, and sometimes far beyond, that at which good results can be expected or required. In some cases this has been due to the specifications under which the filters were installed, and in others, to what must be called an over-rating of the capacity of filters on the part of manufacturers. It is easily possible to force or pass through a filter of given dimensions much more water than it will properly filter, and in view of this it must be expected that there will be more or less yielding on the part of manufacturers placed under competitive conditions to the temptation to over-rate their filters.

Need for Definite, Reasonable Specifications on Capacity.—(9) This condition emphasizes the need and value of a pronouncement on this subject by some such body as the American Society of Mechanical Engineers which will serve for the information and guidance of those who, while having occasional need to specify or use mechanical filters, do not have opportunity to keep fully informed of conditions in that field. It is, therefore, hoped that this report and its recommendations will be of real value to engineers by placing before them information as to what is now the best opinion and practice, and thus enabling them to protect their own work and their clients' interests. To this end your committee most heartily and urgently recommends that when specifying filters there be included not merely the amount of water to be filtered per unit of time, but also specifications as to the rate of filtration per unit of area, or else the area or dimensions of the filter bed. Specifications thus written will insure fair competition and more satisfactory results.

Identity in General Design.—(10) The same general design and the same principle of operation are followed by all the leading manufacturers of mechanical filters, the filtration being downward through a bed of sand superimposed upon layers of gravel, the filters being washed by a reverse flow of water. Competition in construction is, therefore, limited to the excellence of materials and workmanship, to the perfecting of details and to adaptations for convenience in accordance with good filtration engineering practice. While this affords abundant opportunity for conscientious care and requires familiarity with the history of filtration and thorough knowledge and observance of the results of experiments and tests, it does not allow any application of ingenuity to change fundamental requirements that are dependent upon natural laws.

Unnecessary to Standardize Construction Details.—(11) Your committee feels that it would be unwise, at least at this time, to attempt to standardize details of construction, there being a wide range in this field for individual preference or convenience, but there may well be established a standard in regard to the rate of filtration, since the object thereby sought is not mere uniformity but compliance with the limitations imposed by the laws of nature, so that the possible benefits of filtration will be actually and fully realized. It would thus seem to be self-evident, even if it were not fully established by experiment and experience, that the capacity of any filter is dependent upon and determined by two factors: (*a*) The permissible rate per unit of area at which the water can be passed to insure the desired results; (*b*) the effective area of the filter bed.

Agreement Among Leading Filter Manufacturers.—(12) It was made evident by the data gathered that there is a unity of opinion on the part of those best qualified to judge at what rate water may be passed per square foot of filter area to secure desired purification, and that there is a close agreement in the practice of all the leading filter manufacturers in rating the capacity of a filter.

Form of Expressing Capacity.—(13) For convenience, we have expressed the rate of filtration in terms of gallons per square foot of superficial filter bed area per minute, thus combining units of quantity, area and time in a way to make easy the calculation of the amount of water any given filtering unit will properly handle or to estimate the area of filter bed surface that will be required for a given supply. The filtering area should be computed on the upper surface of the filter bed, as the latter lies during normal filtering operation, and no attention should be paid to a greater cross-sectional area such as is sometimes found in horizontal cylindrical filters.

Care as to Maximum Demand.—(14) In deciding upon the size of filters to be installed in any instance, very careful consideration should be given to the maximum flow that will be required at any time, and ample capacity provided. Where the demand is irregular, the maximum requirement is much greater than the average or minimum consumption, and either adequate storage for filtered water should be provided or the rated capacity of the filter made equal to the maximum demand. All filters are capable of passing more than their rated capacity, but beyond certain fairly narrow limits this is always at the expense of the quality of the filtered water, unless more than ordinary care is taken in efficiently coagulating the unfiltered water. As already intimated, the persistent use of moderately high rates above the normal and the occasional use of excessively high rates should be discouraged, if not prohibited.

Depth of Filter Bed.—(15) While in a sense consideration of the filter bed may not be included within the instructions given your committee, we feel that some remarks in this connection will be of value, especially as there seems to be an opinion in some quarters that the use of a thicker filter bed or special methods or appliances for washing, or similar features, make higher rates of filtration permissible. In regard to such points, we would say, that while of course there is a minimum thickness of filter bed that must always be maintained for safety, better results do not follow increased depth. In fact, an excessive depth of sand bed is in some instances objectionable, as it may interfere with proper washing. We find that the minimum thickness of filter bed should be 27 ins., of which at least 18 ins. should be sand or similar fine material. A filter-bed thickness of 33 ins., of which at least 24 ins. is sand or similar fine material of suitable size and grade, is recommended.

Influence of Filter Washing.—(16) While efficient washing of the filter bed must be provided for and while the use of special means or appliances, such as stirrers, air agitation or other means of breaking up the filter bed, may be of value in some cases as means of securing economy in time or of water consumed in washing the filter bed, the direct effect of such means is limited to that secured during the washing process and such effect has no influence one way or the other on the permissible rate of filtration, which is dependent upon and limited by properties inherent in the filter bed itself.

Normal Filtering Material.—(17) The most desirable filter medium is a granular substance of a hard, non-porous, insoluble character, with grains substantially uniform in size and shape, the exact size and uniformity of the particles being open to some variation depending upon local conditions. If properly washed, such a filter bed will remain in efficient working condition for several years.

Special Filtering Material.—(18) Bone charcoal or other porous material is sometimes of aid in the removal of iron, color, tastes or odors. But if they are used it must be recognized that growths of bacteria in the effluent are very likely to occur, although there is no evidence to indicate that such growths include disease-producing germs. These porous media may be used in single or double filtration, as noted in Pars. 20 to 23, inclusive.

Recommended Rate of Filtration.—(19) The permissible rate of filtration in any instance depends upon the character of the water to be filtered and the purpose for which the water is used. If the water is for domestic purposes, whether the filters are installed in a municipal plant or otherwise, the rate of filtration should not exceed that which has been adopted for such service by universal consent of filtration engineers. We therefore recommend that: (*a*) Whenever the water is to be used for domestic purposes or to secure full bacterial purification, the capacity shall be based upon a rate of filtration not to exceed 2 gal. per min. per sq. ft. of filtering area and a coagulant must be used. (*b*) Where a lesser degree of purification is required, either because the water is not to be used for domestic consumption or because the water to be filtered is already sufficiently free from bacteria, or where the filtered water is to be effectively sterilized, a higher rate of filtration may be used, but not to exceed 3 gal. per sq. ft. per min.

Double Filtration in Special Cases.—(20) Your committee finds that there is a limited use made of double filtration; that is, the water is passed through two filters placed in tandem. The consensus of opinion of those consulted and the recommendation of your committee is that when both filters are filled with the same medium this is not the best practice, but that better results will be obtained from the same filters operated in parallel, if they are properly constructed, owing to the slower rate of filtration.

(21) Double or tandem filtration may, however, be used to advantage under some special circumstances, as, for instance, where the filter medium in the second filter is of a very close texture, so as to secure the very highest quality of filtered water by removing fine suspended matters that may pass through an ordinary filter bed.

(22) Double filtration may also be of advantage where the use of a coagulant is not desired or where it is intended to remove iron, color, odor or taste. In such cases sand can be used in the first filter and bone charcoal or similar porous medium in the second. Such practice, however, should be limited to cases where an increase in the numbers of harmless water bacteria, such as frequently occurs in the effluent of a porous filter medium, is not objectionable.

(23) If double filtration is employed, the rate of filtration should not exceed the rate of single filtration, unless warranted by the results of experiments or upon the advice of a competent filtration engineer.

Sterilization.—(24) In earlier years it was frequently the custom to sterilize filter beds with steam, but it was found that the benefit of this treatment was temporary, and it frequently resulted in the growth of water bacteria within the filter. At present, sterilization is normally and preferably secured in the filtered water through the aid of liquid chlorine, hypochlorite of lime, or ultra-violet rays. When properly applied, such treatment will destroy all objectionable bacteria.

Preparatory Treatment.—(25) While this report deals essentially with filters themselves, it is proper to point out that mechanical filters, with the rapid rate of filtration employed, cannot be expected to accomplish the best obtainable results without the securing of proper coagulation; and if the raw water is very turbid, then preliminary sedimentation also must be considered.

(26) In closing this report the committee desires to express its deep loss in the death on August 7, 1915, of J. C. W. Greth, Mem.Am.Soc.M.E., one of the original members, and also its appreciation of his aid in collecting data on the practical state of the art and of his judicially expressed opinion as to the basis of this report.

Discussion.—Jos. W. Ellms: The recommendations of the committee making this report are in agreement with the best modern practice in the design of mechanical filters. They have quite clearly indicated the maximum limits for safe rates of filtration and the minimum depths for the filtering medium. Their conclusions on these points are in accord with the writer's own experience.

In declining to commit themselves in regard to construction details, they are probably justified; yet it might have been well to have enunciated some of the general principles which experience has shown to be essential to the best design. The writer has reference more particularly to underdrain, strainer and waste-trough design. Probably no portions of a mechanical filter are more directly dependent upon proper co-ordination than are those whose principal functions relate to the cleansing of the filter bed. So far as these portions of the filter act as distributors of the influent water to the filter bed, and as outlets for the effluent from the bed, their importance is only of a secondary character.

It is of the utmost importance that the underdrain system of a filter bed shall effect as uniform a distribution of wash water as possible. Whether this is done largely through strainers of various types or through perforated pipes, or in conjunction with the coarse filtering material forming the bottom of the filter bed, is of small consequence, so long as it is actually accomplished. Moreover, a uniform and speedy withdrawal of the dirty wash water by means of properly disposed waste troughs is as essential as uniform distribution of the wash water when entering the filter bed. In other words, a mechanical filter which cannot be quickly and effectively cleansed is defective.

It is to be hoped that the indorsement of the committee's report by the Society will establish certain definite principles of filter design to which manufacturers will conform, and by which purchasers of filters may be guided in judging of the merits of any particular design that may be submitted to them.

George A. Johnson: The report of the committee recommending certain standard procedures in filter practice shows clear evidence that the general proposition has been viewed from the numerous necessary angles and the tentative conclusions drawn from approved practice in filter design and operation. At best the preparation of such a report was a difficult task, for the reason that in water-filtration problems local conditions govern so very largely, and their individual peculiarities are so numerous and varied. This report will serve a valuable purpose in the development of standard practices where possible of application, and the committee is to be congratulated for the skill and conservatism displayed in its preparation.

AMALGAMATION OF McGRAW PUBLISHING CO. AND HILL PUBLISHING CO., OF NEW YORK.

The McGraw Publishing Co., Inc., and the Hill Publishing Co., of New York, have merged. The name of the company will be the McGraw-Hill Publishing Co. James H. McGraw will be president, E. J. Mehren vice-president and general manager, Arthur J. Baldwin vice-president and treasurer. All the papers owned by the two companies will continue as before, except "Engineering News" and "Engineering Record." These two papers will be merged and known as "Engineering News Record," with Charles Whiting Baker as editor and William Buxman business manager.

The new vice-president and general manager of this important combination of publishing houses has done remarkable work since he entered the field of technical journalism. In the minds of those who know E. J. Mehren there is no doubt at all that in this new connection he will, as he always has in the past, demonstrate his fitness for increased responsibility. Mr. Mehren graduated from the University of Illinois in 1906 and joined the staff of "Engineering Record" the following year as associate editor. Six years later, or in 1913, he became editor and under his care "Engineering Record" has shown remarkable editorial development.

In 1916 Great Britain led in shipbuilding with 510 vessels of 619,000 tons. The United States was second with 1,213 vessels of 560,000 tons. Ships built by all other countries numbered 782, of 720,368 tons. Loss to the world's merchant shipping in 1916 through war causes exceeded the total tonnage constructed, according to estimates prepared by the Federal Bureau of Navigation, Washington, D.C. Vessels sunk are put at 1,149, of 2,082,683 tonnage, and those built at 2,506, of 1,890,943 tons. The net reduction was about 200,000 tons, or one and one-half per cent. of the world's total. These figures were gathered from many unofficial sources, but are declared to be approximately correct.

LETTER TO THE EDITOR.

The Quaker Oats Fire.

Sir,—Doubtless many of your readers will be interested in the following facts which are the result of an investigation by the writer following the recent fire at the Quaker Oats plant at Peterborough, Ont.

Below are given some of the records of the investigation, the conclusions drawn, together with a few general remarks and suggestions with regard to fireproofing.

The concrete was badly cracked. The aggregate was gravel. I found when breaking up pieces that many of the smaller as well as of the larger aggregates had a soft layer of varying thickness, (1 to 3 m/m). Some were only partly covered with it. On closer inspection it became evident that these layers were formed from the surface of the stones themselves, and that they were not, for instance, coatings of clay or other foreign material. I picked out some of the aggregates which had these peculiar formations. The coatings were scraped off and analyzed, (see No. 4965). The stones themselves were afterwards ground up and analyzed, (see Nos. 4964, 4966, 4967).

I found, also, in some parts of the concrete, stratified layers of some fine material. A sample of these was collected and analyzed.

Analysis of Aggregates from the Concrete.

	No. 4964.	No. 4966.	No. 4967.
Volatile matters	39.69%	42.56%	41.53%
CaO (lime)	50.64%	54.42%	52.26%
MgO (magnesia)	0.77%	0.58%	0.74%

From these results we see that the damaged aggregates were composed of limestone.

Analysis of the Coating Scraped Off the Above Tested Aggregates.

	No. 4965.
CO_2 (carbonic acid)	25.21%
CaO (lime)	54.32%
MgO (magnesia)	0.86%

The composition of the material is therefore:

CaO, CO_2 (carbonate of lime)	57.29%
CaO, H_2O (slacked lime)	29.38%
MgO, H_2O (hydrate of magnesia)	1.25%

The layers covering the aggregates contained, therefore, considerable amounts of hydrated lime. This was evidently formed in the following way:

The heat from the fire burned the surfaces of the limestone aggregates to quick lime,

$$\underset{\text{Lime-stone.}}{CaO\ CO_2} = \underset{\text{Quick lime.}}{CaO} + \underset{\text{Carbonic acid.}}{CO_2}.$$

This, on the other hand, (CaO) was afterwards slacked to hydrated lime,

$$\underset{\text{Quick lime.}}{CaO} + \underset{\text{Water.}}{H_2O} = \underset{\text{Hydrated lime.}}{CaO\ H_2O}.$$

Part of the lime was slacked when the firemen poured water on the building, and the rest was gradually formed by absorbing moisture from the surroundings. It is, however, of minor importance for the final results in whatever way the hydration took place. The main thing is that when quick lime is slacked the resulting hydrated lime occupies a volume twice to three times as large. The force with which this expansion takes place is so great that it is sometimes utilized for blasting purposes. I found, also, that wherever cracks appeared in the concrete the limestone aggregates were at least partly covered with a layer containing hydrated lime. It is, therefore evident that the cracks in the concrete were in these instances caused by the expansion, which took place when the quick lime formed by the fire was hydrated. The loose structure of the concrete was at least partly due to the same cause. But even the formation of a thin layer of quicklime round the aggregates must injure the concrete, as we know that this substance is much softer and weaker than the limestone. It would in this state act in a similar way as a coating of clay. The concrete would therefore from this cause alone be liable to break down during the fire, without any hydration taking place.

The kind of limestone found in the concrete will burn at about 1,490° F. (810° C.), wherefore the temperature in the effected concrete must have reached at least that point.

It is therefore evident that gravel containing limestone of a similar composition (the approximate B.T.U. was in this case = 7,800) should not be used where a possible fire of the combustible materials, stored in a building, will be able to raise the temperature in the vital concrete construction up to or above 1,490° F. For aggregates containing magnesia limestone the temperature limit is still lower. The poorer the limestone the easier will it "burn," but the less is the expansion when it hydrates. If the aggregates in this special case had been of a silicous character, as, for instance, granite, trap or other similar substances, the concrete would have, without doubt, withstood the fire, if not of too long duration.

The thermal value of the main part of the material stored in the building was found from actual test to be 8,092 B.T.U.

It was pointed out above that the loose structure of the concrete was due to the expansion caused by the hydration of quick lime. I found, however, indications that the concrete had been worked too wet. We know, however, that too much water will weaken the structure and make it porous and less able to take care of the different stresses. The above-mentioned strata found point to a mixture too high in water. The analysis of them gave the following results:

No. 4968.

Volatile matters	18.21%
SiO_2 (silica)	19.00%
Al_2O_3 (oxide of aluminum)	6.88%
Fi_2O_3 (oxide of iron)	1.43%
CaO (lime)	50.83%
MgO (magnesia)	2.32%
SO_3 (sulphuric acid)	1.29%

The composition of this material in the dry state would be as follows:

Volatile matters	1.46%
SiO_2 (silica)	22.84%
Al_2O_3 (oxide of aluminum)	8.29%
Fi_2O_3 (oxide of iron)	1.72%
CaO (lime)	61.24%
MgO (magnesia)	2.79%
SO_3 (sulphuric acid)	1.55%

From the analysis we may draw the conclusion that the stratified material was Portland cement. This circumstance indicates, also, that the mixing of the concrete might have been done better.

Porous and weak concrete is also caused by lack of cement. In this special case I do not know of any method by which we can determine the proportions of cement to aggregates. I analyzed the concrete and the results are:

No. 4983.

H_2O (organic matters)	4.10%
CO_2 (carbonic acid)	23.18%
SiO_2 (silica)	22.18%
Al_2O_3 (oxide of aluminium)	5.45%
Fi_2O_3 (oxide of iron)	1.56%
CaO (lime)	41.04%
MgO (magnesia)	1.34%
SO_3 (sulphuric acid)	0.24%

The CaO factor = 4.82 cc n/2 HCl.
Sp. gravity, 2.6211.

The only conclusion I can draw in this special case is that the concrete mixture was poorer than 1:6. I may add that, estimating the proportions from three different points of view gives the same result.

From the results obtained in the above outlined investigation I draw the following conclusions: The concrete suffered from the fire because the limestone aggregates were on their surfaces through the comparatively strong heat changed to quick lime, and this in its turn was hydrated when acted upon by water. The resulting great expansion caused the cracks and the loosening-up of the concrete.

In connection with this subject I may be allowed the following remarks and suggestions, based on the above findings:

There is in the public mind a rather uncertain idea as to the property of a fireproof structure. Some seem to think that it is fireproof if the materials in the same are not subject to combustion and others, that they must not only be inflammable, but that they should also endure fire from other materials without injury. There is, however, no building material in general use that is immune from the intensity of heat possible from combustion. The second definition in its fullest meaning must therefore be thought of more as an ideal than a practical possibility. The first view, on the other hand, includes some building materials which, while they do not burn are easily damaged by fire. Would it not, therefore, be well to go between the two and consider them as the lower and upper limits? Looking at the subject from this point, a fireproof building would be a construction of non-combustible materials that will endure the influence of fire up to a certain limit without being damaged. The limit or degree of heat it might be called upon to withstand will depend on the nature and quantity of combustible materials stored inside the same or in its immediate surrounding, and may therefore be estimated beforehand by the architect. By approaching the subject from this point of view the meaning of the word "fireproof" becomes more elastic and assumes an appropriate degree of individuality for each special case. It gives an adequate protection to the owner, and it will also serve to prevent useless expenditure, as the most suitable material can in each instance be selected to fit that special case.

Concrete has in many a fire proved its good qualities, and it would have done the same in this special case if the proper aggregates had been selected to suit the probable conditions.

Hoping the above will serve a good purpose.

A. G. LARSSON, C.E.

St. Mary's, Ont., February 12th, 1917.

Editorial

STANDARDIZING WOOD BLOCK PAVING SPECIFICATIONS.

Last year a committee was appointed by the American Wood Preservers' Association for the purpose of collecting and compiling all specifications and definitions covering wood block paving and to recommend such revision as may be necessary and present for insertion in the Manual. This committee was furthermore instructed to prepare a specification for the treatment and laying of creosoted wood blocks. The committee confined its specifications to the superstructure of a roadway or floor and included the surfacing of the substructure, the cushion, the blocks and filler.

It had on it representatives of the following associations: American Wood Preservers' Association, American Society of Municipal Improvements, American Society for Testing Materials, American Society of Civil Engineers, American Railway Engineering Association and the Southern Pine Association. The object was to endeavor if possible to have the committees who were working on wood-block paving specifications present identical specifications to their respective associations. In this success was attained, the members of each of the committees agreeing to present identical specifications on timber, size of blocks, treatment, handling after treatment and inspection.

The adoption by these various associations of uniform specifications is undoubtedly a step in the right direction.

THE PUBLIC AND THE HIGHWAY ENGINEER.

The season of the year is here when the many problems connected with the design, construction and maintenance of our highways force themselves upon those who are directly and indirectly concerned.

During the next few months a number of organizations, the principal object of which is to further the interests of good roads, will hold their annual meetings.

This week the Ontario Good Roads Association is holding its annual meeting in Toronto. At this meeting representatives from all parts of Ontario gather and discuss various phases of highway engineering.

From March 27th to 30th inclusive, under the auspices of the Ontario Department of Highways, a most important conference will be held. This conference will concern itself more specifically with the administrative and practical side of road building and is bound to be of great assistance to all those engineers and superintendents who are fortunate enough to be present and take part in the discussions.

The Fourth Canadian and Dominion Congress is to be held at Ottawa from April 10th to 14th inclusive. This convention, in turn, whose object is the spreading of the gospel of good roads, makes its appeal to a wider circle, attracting delegates from practically all over Canada and also the United States.

It is gratifying to find such an intelligent interest being taken in the work of such organizations as those named. They do a general educational work and make a real contribution toward a more intelligent and genuine spirit of co-operation between the layman and the highway engineer who is more directly concerned with the actual design and construction of the roads.

Highway engineers, because of the nature of their work, are brought more prominently before the public than engineers engaged in many other lines. At least it is probably true that more people are interested in road work than in the majority of other public enterprises, for the reason that all have occasion to use the highways. The condition of the roads in any community is generally a matter of personal concern.

The work of the highway engineer is more often the subject of discussion than are other branches of engineering. Such discussion is not always intelligent and there is much need for a great deal of activity on the part of highway engineers and organizations such as those referred to, in bringing this branch of engineering work before the layman in such a way as to make clear some of the points upon which he might be better informed.

Whatever can be done in the direction of molding public opinion to support wise plans and defeat unwise ones will be a service to the public as well as to the profession of highway engineering.

WORK OF THE ADVISORY COUNCIL FOR SCIENTIFIC AND INDUSTRIAL RESEARCH.

The Advisory Council for Scientific and Industrial Research, of which Dr. A. B. Macallum is chairman, has just issued a very important review of the subject, following a conference which was recently held in Ottawa. Some forty projects, each bearing on vital phases of scientific conservation and development of Canada's natural resources, have been submitted to the council.

Some of the larger projects now in view include a comprehensive industrial census, the training and utilization in industrial establishments of "efficiency experts," the creation of technical laboratories under State co-operation at the great industrial centres to give free help to manufacturers in solving their problems, the utilization and development of the latent fuel resources, particularly of the Prairie Provinces, and the preservation of the diminishing timber resources of Eastern Canada.

The council will issue questionnaires to the manufacturers, the technical societies, the various government departments, and the universities of the Dominion, asking for information with reference to the laboratories and various other agencies of research now in operation in the Dominion; the men now engaged in or available for research work; the raw materials required for our industries; the by-products produced but not at present utilized; and other matters required in the development of its work. In securing this information the council will work in close co-operation with the manufacturers' associations and the various technical societies of the Dominion. It is expected that the replies to the questionnaires will show many lines along which the council may assist in the development of Canadian industries.

The council will recommend the establishment of twenty or more studentships and fellowships in our universities and technical schools, which will be given to men who have completed their regular course of study and have displayed a special aptitude for scientific research. These will enable such men to pursue a course of advanced work at college for a further period. Arrangements will also be made by which men after graduation will be placed in one or other of the great manufacturing establishments of the Dominion, where they will continue their training under the conditions of actual commercial practice.

In order to furnish direct assistance to the manufacturing industries of Canada at once the council is recommending the establishment at certain of the great industrial centres of the Dominion, such as Toronto, Montreal and Winnipeg, in co-operation with the Provincial Government or other bodies, of Industrial Research Bureaus, where a complete set of technical magazines and trade journals will be found, and where technical staffs, provided with suitable and properly equipped laboratories, will assist the manufacturers of the district in solving problems which present themselves in their factories or works.

In addition to these broad general movements for the advancement of the industries of the Dominion, the council has decided to examine carefully a number of specific projects which have been submitted to it, and which appear to give promise of yielding valuable results. Among these latter one may be mentioned.

This has for its object the provision of an adequate supply of good fuel for the western plains, more especially in the provinces of Saskatchewan and Manitoba. There are in the former province large supplies of lignite. This is an inferior fuel, possessing a relatively low heating power, and which, furthermore, will not stand shipment and storage. It is, therefore, of comparatively little value for domestic or manufacturing purposes. The council, however, believes that by a special treatment there may be produced from this lignite two grades of high-class briquetted fuel, one similar to anthracite or hard coal in character, and the other resembling soft coal in general character; and at the same time certain very valuable by-products may be secured. The Department of Mines and the Commission of Conservation have already carried out a good deal of investigation in connection with this problem, and the former department is now making some further studies for the council. If they give satisfactory results, the council will advise that an experimental plant to turn out this high-grade fuel on a commercial scale be erected, and the possibility of producing this fuel at a cost considerably lower than that at which coal from the United States is now laid down in Manitoba and Saskatchewan be demonstrated on a large scale and the coal actually placed on the market. With an abundant supply of good, cheap fuel, the conditions of life on the great plains in winter will be much improved.

A report has been circulated to the effect that the plant of the S. Morgan Smith Company, York, Pa., had been taken over by the Bethlehem Steel Corporation. We are informed that this report is incorrect, there being absolutely no foundation for it.

PERSONAL.

R. H. SPERLING, formerly general manager of the British Columbia Electric Railway Company, has been granted a commission in the British navy.

G. GORDON GALE, formerly general manager and chief engineer, Hull Electric Company, has been appointed vice-president and general manager.

EDWIN H. VERNER has been appointed municipal engineer for Langley, B.C. He formerly held a similar position for Port Coquitlam and Coquitlam Municipality.

F. E. FIELD, M.Can.Soc.C.E., the engineer who has represented the city during the construction of the Montreal filtration plant, has been appointed superintending engineer in anticipation of the early operation of the plant.

J. M. WOODMAN, formerly superintendent of the C.P.R. terminals at Winnipeg, has been transferred to Montreal. R. C. MORGAN, formerly divisional superintendent at Fort William, will succeed him, and A. F. HAWKINS, of Moose Jaw, will go to Fort William.

OBITUARY.

RICHARD BELL, for the last twenty-three years chief engineer of the Sarnia, Ont., pumping station, died on February 19th from injuries received on January 23rd at the George Street station. Mr. Bell was 43 years of age.

PATRICK TALBOT BOWLER, who for almost a quarter of a century was city electrician of New Westminster, B.C., died recently following an operation. He had been in failing health for several years and resigned his duties early in 1916.

ASSOCIATION OF ONTARIO LAND SURVEYORS.

At the twenty-fifth annual convention of the Association of Ontario Land Surveyors, held last week at the Engineers' Club, Toronto, the necessity of a more comprehensive plan of highway construction in Canada, in the interests of national prosperity, was pointed out by Vice-President James J. MacKay, of Hamilton, in a paper which he read on "Good Roads."

The proceedings were closed by a luncheon which was attended by veterans of the association, who had held certificates since before Confederation, after which many of the members availed themselves of an offer to inspect some munition factories.

Officers elected are: President, James J. MacKay, Hamilton; vice-president, H. J. Beattie, Pembroke; secretary-treasurer, L. V. Rorke, Toronto.

The first central station in Japan was opened in 1887 with a 75-lamp dynamo supplying Tokio. The Lake Inawashiro plant has now six 10,000 horse-power turbines and transmits power 140 miles to Tokio at 115,000 volts. At the end of 1914 there were 1,940 stations generating electricity, 390 being central stations, 24 railway plants, 47 combined railway and central station plants, 1,366 isolated plants, and the remainder official plants. Water power is used in 695, steam in 788, and gas in 457 of the stations. The total capacity in 1914 was 608,554 kilowatts, of which 341,809 kilowatts was in central station plants, and 140,022 kilowatts in isolated plants. Water-power equipment totalled 366,243 kilowatts; steam 217,967 kilowatts; and gas 24,344 kilowatts. There were 21,909 miles of aerial, and 751 miles of underground transmission lines.