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THE ROLLING AND FLOATING STEEL CAISSONS OF THE LEVIS DRY DOCK AT LAUZON, P.Q.*

PART I.

A DETAILED DESCRIPTION OF THE DESIGN, FABRICATION AND ERECTION OF THE ROLLING CAISSON.

By LESSLIE R. THOMSON, B.A.Sc., A.M.Can.Soc.C.E., Assoc.M.Am.Soc.C.E.,
Engineering Staff, Dominion Bridge Co.

SEVERAL features in connection with the new government dry dock at Levis, P.Q., are very interesting, presenting as they do some rather unique ideas relative to marine work. In this paper, which has been read before the Canadian Society of Civil Engineers, it is the intention to describe only the caissons or gates, although it will be difficult not to stray from this object in attempting to make certain parts the clearer.

The problem which presents itself at the mouth of a graving dock is familiar to the engineering profession. Whatever device is used to close the entrance, must be capable of being swung in or out expeditiously, be able to withstand the hydrostatic load when the berth is unwatered, and also allow the sea water to enter through it when the berth is filling. By this means the dry dock is very quickly filled and a great deal of time is saved. The problem is seen at once to be very similar to that occurring at a canal lock, and it is owing to this similarity that so many of the early graving docks and even some later examples were equipped with mitred gates. There are, however, certain disadvantages connected with the use of mitred gates, such as the length of wall absorbed in housing them when the berth is open, and the absence of a communicating bridge for wheeled traffic when the berth is closed. The use of a rolling caisson, on the other hand, involves neither of these disadvantages, for it is housed in a recess lying transversely to the berth and a communicating passageway is always provided along the top as its width easily admits the economical construction of a folding bridge of some type.

The cost of either type of caisson will usually exceed that of a pair of mitred gates, but is, on the other hand, less than a pair of gates plus a small swing bridge.

The relative suitability of these various schemes to close dock entrances has been summed up by Mr. W. G. Wales in Proc. Inst. C.E., Vol. cxxii., as follows:—

"In conclusion, ship-caissons are adapted for dry docks, and for locks and entrances in sheltered and non-tidal positions; sliding caissons for locks and entrances in tidal and sheltered positions; and dock gates for entrances in exposed positions and for commercial docks."

The layout of the new dry dock is shown in Fig. 1. The main dimensions of the berth are: Length, 1,150 feet; width at top, 144 feet; clearance width at rolling caisson sills, 120 feet; depth of water over sills at mean water level, 25 feet; at extreme high-water spring tides, 50 feet.

At about mid-point of the berth will be noted the bearing sills lettered "C." These are the sills for the floating caisson and are to be used when it is desired to dock vessels less than 650 feet long. Owing to this arrangement it is not necessary to unwater the whole dock for small vessels. Near the outer extremity are seen the sills for the rolling caisson and its recess chamber, while still further out may be seen a pair of bearing sills lettered "A." These are duplicates of those at "C," being moulded to exactly fit the floating caisson. Hence the floating caisson may be used to close either half or the whole of the berth. In the latter capacity it may serve, too, as an emergency gate during a breakdown of the rolling caisson or any part of the interior of the dock, allowing the berth to remain unwatered during all repairs. Consequently, by reason of this layout, two caissons—one rolling and one floating—are able to close one-half or the whole of the berth, as desired, and also supply an emergency gate when necessary. This is a distinctly economical arrangement.

The large dimensions of these caissons are a little difficult to realize inasmuch as the gates are among the largest of their kind in the world. The floating caisson is larger than that at the new Ferrol Yard, Spain, and also longer than the one for the Panama Canal, though not quite so deep. Hence, with a dock length of 1,150 feet and a width of 120 feet there is no danger but that the largest boats that are even contemplated at present, may be berthed with ease at the new Levis graving dock.

In designing the rolling caisson it was necessary to take into consideration the severe climatic conditions to which it would be subjected. The caisson itself consists of a fabricated steel gate 123 feet long, 19 feet 3 inches broad and 46 feet deep. All elevations of this gate are rectangular, and all horizontal sections are trapezoids and similar in outline. The hydrostatic loads are carried to the vertical ends by means of two trusses and one plate girder, and a small proportion to the bottom sill by the skin stiffeners running between the lower truss and the bottom. It is important to grasp at the outset that the water loads are taken first by skin plates to stiffeners or ribs, and thence to two horizontal trusses and a girder;

*Extract of a paper read before a meeting of the Mechanical Section of the Canadian Society of Civil Engineers at Montreal, March 30th, 1916.

and it is these latter that are carrying the real loads of the caisson.

The girder serves to divide the interior into two water-tight compartments, the upper one to be described as the tidal chamber and the lower one to be known as the ballast chamber. This division is placed at a height of 23 feet 6 inches above the bottom of the caisson, and marks the approximate load water line of the gate. Consequently, under normal conditions the large weight on the rollers is greatly relieved by the buoyancy, and the power necessary for rolling the gate is thus materially reduced. As the tide rises above this 23-foot line, it is allowed free access to the tidal chamber, and in this way the complete flotation of the gate is prevented by the automatic introduction of the requisite amount of water ballast. The foregoing is the operation during the summer. In the winter no water is allowed in the tidal chamber, but the ballast chamber is completely flooded and the enclosed water, heated by steam, serves to counteract the buoyancy of the caisson.

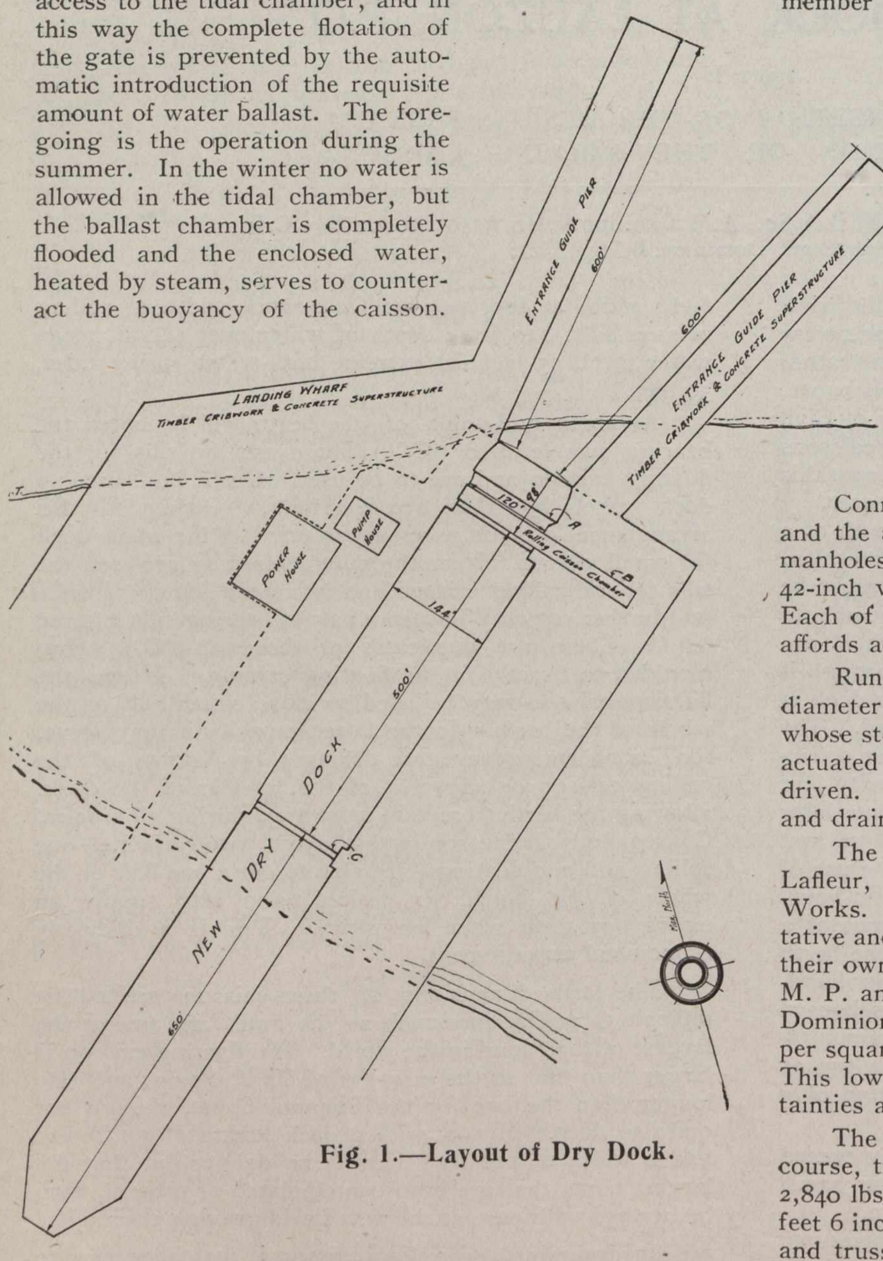


Fig. 1.—Layout of Dry Dock.

The gate is operated by means of flexible cables fastened to each end of a large yoke attached to the inner end of the caisson, which during its longitudinal movement travels on two lines of steel rollers set in the floor of the berth at about 8-foot centres. These rollers bear upon two steel rails on the bottom of the gate, each rail being 6-inch x 9-inch sections of solid medium hard steel, in lengths of about 15 feet. The seal is accomplished by the hydrostatic pressure forcing the caisson against the sills, the actual bearing pieces being 7-inch x 18-inch white oak strips on vertical edges, and 7-inch x 12-inch white oak strips on the horizontal or bottom line.

The skin plates are ¾ inch on the bottom and sides to the height of about two-thirds of the caisson, when ½-inch plate was used. All horizontal seams are single lap joints, all vertical seams are butt joints (D.R.). The vertical skin plates are stiffened by short I-beams and angles placed at 2-foot 10-inch centres, running between bottom and truss A, truss A and girder, girder and truss B, and truss B and top.

The vertical sway bracing is placed between the trusses and the girder, and between truss A and the bottom at every panel point (17-foot centres) except at the top where it is placed at 8-foot 6-inch centres. Each member consists of two 6-inch x 4-inch x ½-inch angles with gussets between for connection to the main material.

The horizontal lateral bracing is placed at the top of the caisson only, at the level of the operating sidewalk. It is designed to provide rigidity against any racking effect that might result from one corner coming into violent contact with the sill.

At the top of the caisson is a folding bridge designed to carry traffic between the two sides of the berth when the caisson is in place, but so arranged that when the caisson is in its recess the bridge is completely folded underneath the roof of the chamber. The operation of this bridge is automatic.

Connection at all times between the ballast chamber and the atmosphere is maintained by means of two oval manholes, each large enough to permit one of the main 42-inch valves being lifted through it to the outside air. Each of them is provided with a ladder inside and thus affords access for inspection, etc.

Running from face to face of caisson are six 42-inch diameter filling culverts controlled by 42-inch gate valves whose stems are taken up to the operating level and there actuated mechanically by a horizontal countershaft, motor driven. There are, also, hand-operated valves for filling and draining the tidal chamber, ballast chamber, etc.

The rolling caisson was designed primarily by Eugene Lafleur, chief engineer of the Department of Public Works. The design proposed, however, was only tentative and the tenderers were given the privilege to submit their own designs. The successful bidders were Messrs. M. P. and J. T. Davis, who turned the work over to the Dominion Bridge Company. A unit stress of 12,000 lbs. per square inch was adopted for the structural steel work. This low stress was in order to take care of any uncertainties as to loading or unavoidable eccentricities.

The loading of the whole gate was assumed, of course, triangular with a unit pressure on the bottom of 2,840 lbs. per square foot, corresponding to a head of 45 feet 6 inches of water. The centres of truss A, the girder and truss B were fixed as 12 feet 6 inches, 11 feet, 10 feet and 12 feet, and on reference to Fig. 2 these are seen. The water pressure of each main division was assumed to act through its own C. of G. and the position of this point determined the proportion that was distributed to each truss or girder. The only point about which any interest might centre is the loading of truss B. It would clearly take its due proportion of W-3 and, owing to the absence of any support at the top of the gate, all of W-4. In addition to these quantities, however, there is an overturning effect due to W-4 that must be resisted. The location of W-4 is 4 feet above truss B; hence, bending moment is 4,500 × 4 = 18,000 lbs. per lineal foot. If this moment be taken up entirely between truss B and the girder (and this is the assumption that places the heaviest load on B), the extra load on the truss is 18,000 ÷ 10 = 1,800 lbs. per lineal foot. The direct load described above

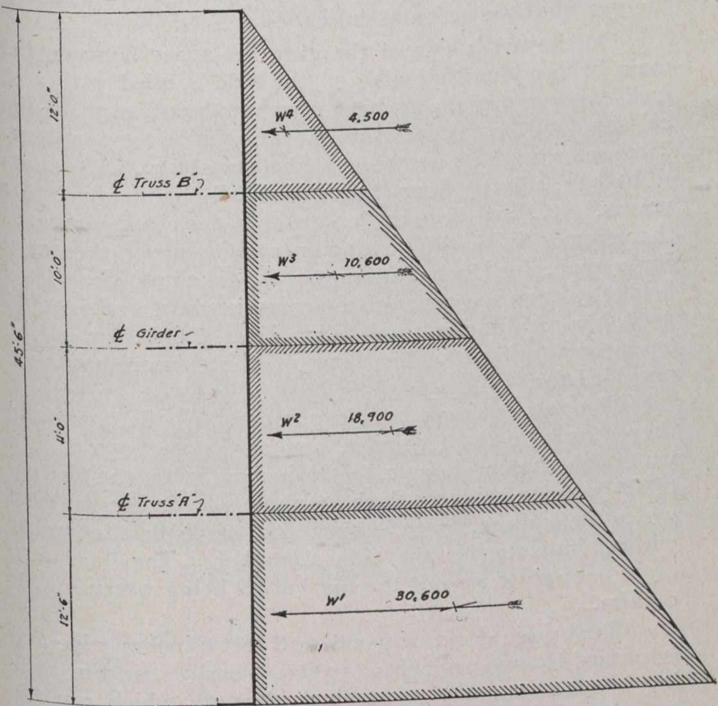


Fig. 2.—Hydrostatic Loading Against Rolling Caisson.

is 9,280 lbs. per lineal foot. Therefore, total load on truss is 11,080 lbs. per lineal foot.

The load on truss A is 24,600 lbs. per lineal foot. As mentioned previously, the unit stress specified was 12,000 lbs. per square inch, and this was interpreted by all parties to mean 12,000 lbs. per square inch tension and the same for compression, unless the Dominion Government Specifications '08 formulae would give less when reduced for $\frac{l}{r}$ ratio.

In summer, the compression chord of truss A would carry the water load between panel points and in winter the tension chord would do the same with, of course, a much smaller head. Owing to the full splicing of the chords the maximum bending moment assumed in them, due to flexure, was $1/10\ w\ l^2$. Preliminary calculations were made on two or three sections of both the compression and tension chords, and average values for extreme flexural fibre stress were assumed in order to fix the exact sections. The average amount deducted from the compression chord, including end post, was 3,200 lbs. per square inch. The $\frac{l}{r}$ ratio for all these sections was so small that no reduction of the 12,000 lbs. became necessary.

In the design of truss A very little need be mentioned except the large sizes of the members and gussets. The chords of this truss are the largest simple truss chords that have ever been built by the Dominion Bridge Company—attention being called to the fact that the gross area of the tension chord in truss A is 342 square inches. The magnitude of this may be realized when one recalls the fact that the cross-sectional area of the centre chord of one of the big Lachine trusses is 302 square inches and that of the lower chord of the St. John arch is 347.5 square inches. In the detailing, every precaution was taken to see that the rivets were capable of developing in each group the requisite amount of stress. The value assumed for rivets stressed in two directions simultaneously was adjusted accordingly.

The horizontal load on the girder is 14,600 lbs. per lineal foot and nothing has been taken from this figure to allow for the negative loading induced in it by the overturning effect of the upper water. The girder is stiffened underneath by 24-inch @ 80 lbs. I's in order to carry the load of tidal chamber when full of water. The cover plates are run far enough beyond their theoretical length to take up their value.

Owing to the magnitude of the loads and to the difficulty of developing the full stress in those portions of the cover plates and skin plates that act as flanges, it became essential to use 1-inch diameter rivets in all main connections of truss A and the girder.

The load on the upper truss B, as mentioned before, equals 11,080 lbs. per lineal foot, and in this truss no difficulty was experienced with the capacity of the rivets, hence ⅞-inch diameter rivets were employed. As in truss A, the working stresses used in the main chords and end post were reduced for flexure by the following amounts: Compression chords, 3,400 lbs. per square inch; end post, 3,000 lbs. per square inch; tension chord, 3,100 lbs. per square inch.

The distribution of the loads bearing on the vertical sides became at once not only very important but also one of the most difficult problems met in the design of the whole gate. The reactions were: Truss A, 1,512,000 lbs; girder, 900,000 lbs.; truss B, 680,000 lbs. The reaction of truss A, if spread over a length of 11.75 feet (½ of 11 + ½ of 12.5 = 11.75 feet its own proportion) and a width of 18 inches would give an average concentration of about

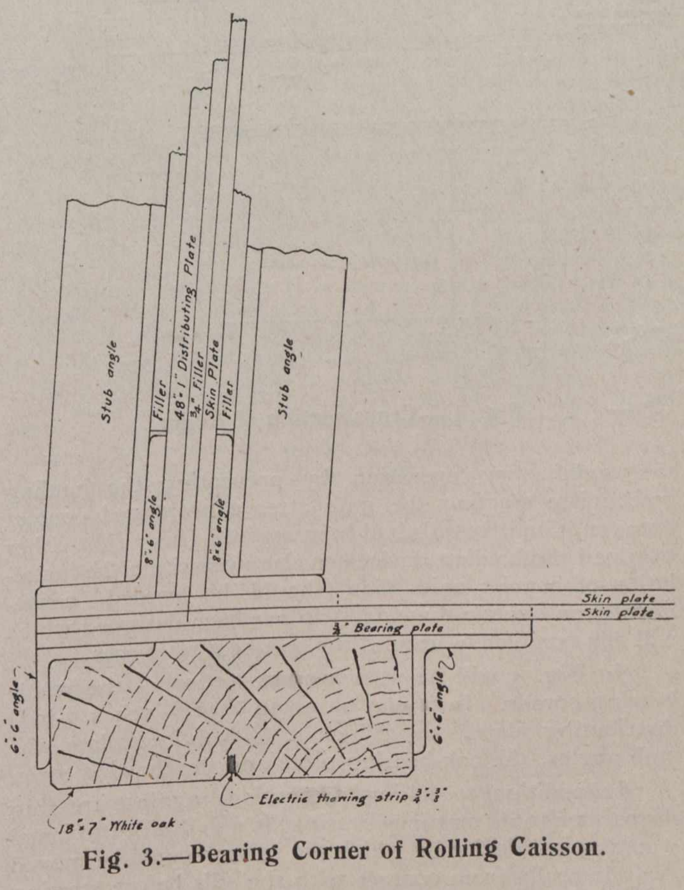


Fig. 3.—Bearing Corner of Rolling Caisson.

86,000 lbs. per square foot—600 lbs. per square inch—which is even then very large for sill pressures. It was felt that, though the width of the caisson was fairly good (19 feet) the end itself would, owing to its construction, be hardly stiff enough to really distribute the highly concentrated load of truss A. Various studies were made to attain some thoroughly practical arrangement by which the load could be spread. After careful consideration being given to several schemes, among which may be mentioned a series of radiating struts from the outer extremity of the end post, it was decided to place along the whole bearing surface a 48-inch x 1-inch continuous distributing plate whose edge would be faced to bear on the filler or skin plate lying behind the 18-inch x ¾-inch bed plate between the caisson and its seal-strip angles. This plate was reinforced at varying centres on the outside by pairs of stub angles 5 inches x 3½ inches x ¾ inch, about 4 feet long. On the inside, similar stub angles were placed at frequent intervals. The object of this whole construction was to insure, if possible, that the load, undoubtedly having a tendency to enter the skin plate, would in turn be taken from these skin plates to the long reinforcing plate because of the greater stiffness of the latter due to its continuity. This quality in the reinforcing plate would then, of course, assure the distribution of the load to the oak and thence to the sills.

It was felt that the question of the freezing of the caisson to the sills might seriously interfere with

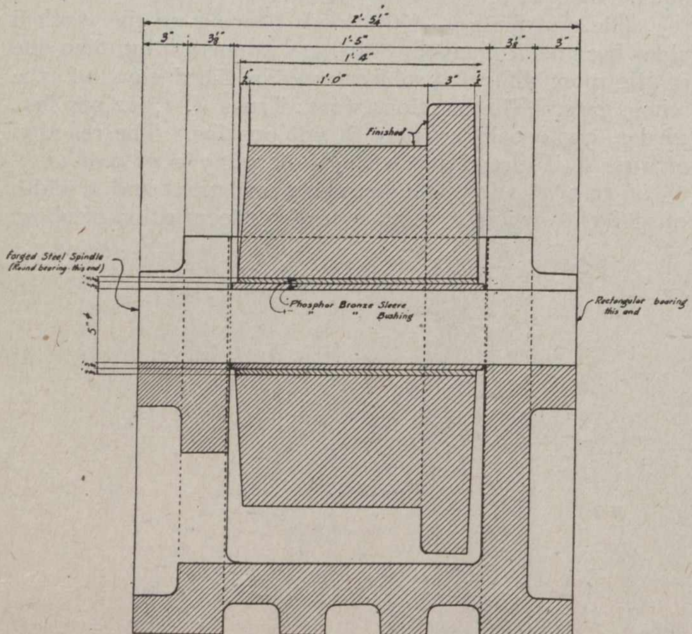


Fig. 4.—Cross-section of Roller.

the satisfactory operation by preventing the hauling mechanism starting the gate after a prolonged contact between it and the sills had been made. This might either overload the hauling devices or else necessitate the installation of heavier units than should be needed, consequently an electric thawing strip has been designed which will run completely around the oak-bearing pieces.

In Fig. 3 will be seen a cross-section of the whole bearing corner. In it can be noted the 48-inch x 1-inch distributing plate, the 18-inch x ¾-inch bed plate, the stub angles, the oak-bearing piece and the thawing strip.

In addition to taking precautions against freezing the caisson to its masonry bearing, it was decided to instal a mechanical device by which the caisson could be moved away laterally from contact with the sills before starting to roll it inwards. By this means no heavy frictional load due to the intimate nature of the contact between caisson and walls would be thrown on the hauling mechanism. In addition, this device could be used to push the caisson against the sills preparatory to its being used as a stop gate. This would prevent the violent jar that would inevitably result were the gradually increasing hydrostatic pressure allowed to finally and suddenly overcome the static friction between the gate and its rollers. Hence a toggle arrangement was designed capable of performing the two above-mentioned duties.

On the berth side of the gate the space between the main bridge longitudinals and the side is filled with 5/16 inch checkered plate. This runs the whole length of the caisson, and constitutes the operating platform or deck. The motor, with its control and rheostats, etc., in a watertight box, all the floor-stands and all hand-wheels for valves, etc., are brought up to this deck and operated therefrom. With this arrangement it is never necessary for a man, for operating purposes, to climb from the bridge to the open parts of the caisson where a slip would probably result in a fatal fall. Thus the safety of the operators during the handling of the caisson has been provided for.

The manholes are two in number. At the lower end they terminate at the girder, and at the upper end, in a watertight hatch about 2 feet 6 inches below the bottom of the wooden bridge. Each manhole affords access, even when the caisson is in service, to the ballast chamber without unwatering the tidal chamber. They are also large enough to permit the big valves being taken up for repairs.

The bridge deck, of wood, is 8 feet 6 inches wide and is carried by two 10-inch @ 35 lbs. I-beam stringers resting in small supporting castings through which run the floor beams of round cold-rolled steel shafting 2¾ inches in diameter. These floor beams lead into the bridge posts, which are flats with forged eye heads at points of floor beam attachment and also at their lower extremities where they are fastened to longitudinal supporting stringers of 10-inch @ 20 lbs. ship channels with flanges turned inwards. This fastening is such that the post may swing in vertical arcs through an angle of about 80 degrees. The upper extremities of these posts are fastened by pivoted connections to fence railings of 3-inch x ⅜-inch flats. Thus the whole system may, owing to absence of stiffening bracing, swing downwards until it lies completely under the top or clearance line of caisson. When in this position the caisson may roll into its recess with no post above the recess cover. In order that the bridge shall not be free to fold by gravity only, counterweights are supplied in three sets of bridge posts which are then extended below the ship channels to receive the counterweight box. The extensions are accomplished by means of special posts of 12-inch @ 25 lbs. channels.

As the only occasions when the bridge must be folded occur when it is in its recess, it was decided to make this folding automatic.

The skin plates are designed to figure as small strips of unit width acting as continuous beams over a large number of supports $\frac{1}{12} \cdot wl^2$ being the moment formula used. No attempt was made to combine bending stresses in skin plates in the vicinity of the trusses with the main chord stresses induced in them in virtue of certain portions being taken as chord material.

The draw bar pull is exerted on the end of the caisson by means of a large bracket bolted with twelve 1¼-inch diameter bolts, and knee-braced below. These bolts are 3 feet ½ inch long and, passing through the skin plates, engage a steel anchorage that thoroughly distributes the load. By means of a 15-inch horizontal channel a certain proportion of it is conveyed to a number of the small vertical channels, but the major portion of the draw bar pull is taken by four 8-inch x 6-inch angles to the top sides of the caisson.

The bottom of the caisson is stiffened by 24-inch @ 80 lbs. I's placed at 2-foot 10-inch centres and has also about 6 feet of solid concrete ballast which, of course, aids in distributing the loads delivered by the 6-inch x 9-inch steel rails.

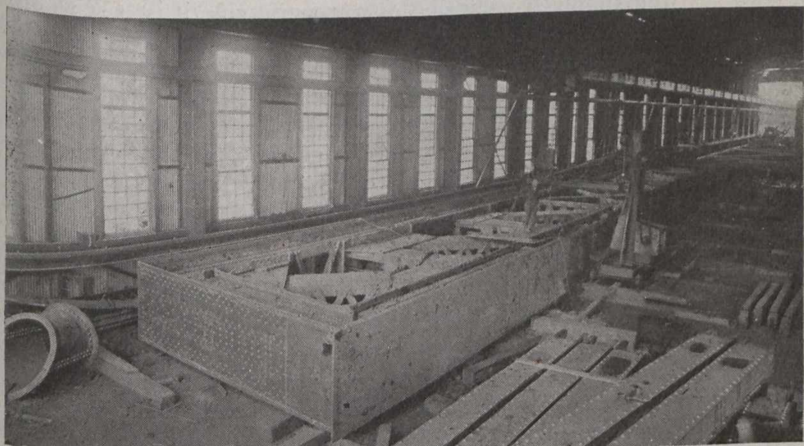


Fig. 5.—Truss "A" Assembled in Shops.

The rollers for this caisson are of solid steel 2 feet in diameter with a flange 3 inches x 3 inches, and a face width of 1 foot. The thickness along the bearing is 16 inches. Fitted on the inside of the rollers is a phosphor bronze bushing 16 inches long and 3/8 inch thick. This bears on a phosphor bronze sleeve 16¾ inches long and also 3/8 inch thick. This sleeve in turn fits over and bears upon the forged steel spindle 5 inches in diameter. One end of this spindle is forged square to prevent it turning in its bearing. The frame for the bearings (one frame per roller) is cast iron, of heavy design with its outer surface deeply indented to make an intimate bond with the concrete. These details are exhibited in Fig. 4, which shows a cross-section of one roller and frame.

The fabrication of this caisson proceeded along the usual lines for first-class bridge work. Complete detail drawings were prepared and wooden templates made.

As will be remembered, truss A and the girder were designed for 1 inch diameter rivets and truss B for 7/8 inch diameter rivets. In order to keep all punching the same it was decided to punch everything 15/16 inch. This would be, of course, sub-punching for truss A and the girder, and full size for truss B. In the former cases the holes would be reamed to 1 1/16 inches for the 1 inch rivets.

In order to be sure of accurate fitting, truss A with its skin plates, and parts of the girder were assembled in the yard and the reaming was then done. Fig. 5 shows the truss A being reamed. A good idea of the weight and size of the members may be obtained by comparing the depth of the chord with the man beside the reamer. The magnitude of the heavy gussets may also be noticed.

The hydraulic features consist of six main culverts used to flood the berth; four valves to let sea water into tidal chamber; two valves to let sea water into ballast chamber; valves to drain the superfluous water off the tidal and ballast chambers. The six main culverts are of 42-inch diameter steel lap riveted section, terminating in forged flanged ends riveted to the sides of the caisson and bolted to the flange of the main gate valves, whose vertical outer lines are located 3 feet 10¾ inches from the berth side of the caisson.

The inlets for sea water in the tidal and ballast chambers are operated simultaneously. To flood the tidal chamber there are four 10-inch valves with horizontal stems situated 3 feet ¼ inch below the girder deck. Two 10-inch valves with vertical stems are used to flood the ballast chamber. All the main culvert valves are electrically operated. To drain these chambers there are two 6-inch valves in each, which empty into the ship berth. It was decided that the rolling caisson should be erected first as it could be used as a gate to protect the ship berth during construction. The contractors gave the Dominion Bridge Company permission to use the excavated part of the mouth of the berth as an erection site.

The rolling caisson was erected lying transversely across the berth, with its longitudinal centre line coincident with the centre line of its recess chamber. The floating caisson was also built here. Between the two caissons a large timber erection trestle was built to carry the erection car.

Before the Dominion Bridge Company started steel erection the contractors had placed the rollers in the floor of the berth. As it was necessary to caulk the seams of the caisson it was propped up on jacks 3 feet 6 inches above the floor of the ship berth and assembling was completed up to truss A. No great difficulty was experienced beyond that caused by ice and snow getting in the rollers.

At the recent annual meeting of the Commission of Conservation, a resolution was passed requesting the various provincial governments to take steps to secure complete reports of all losses from fires occurring within their boundaries, and the extent, if any, to which the property was insured.

A new invention has been patented in the United States the object of which is the production of timber, particularly piles, which shall be fully preserved for a part only of their length, that is, at the part where they are subject to decay. The method consists in first removing the outer bark from the entire length of the stick, and removing the skin, or inner bark from only that part which it is desired to impregnate with the oil under the usual or any preferred conditions of heat, vacuum, pressure, etc. Wood does not decay in that part which is permanently in wet soil, but the part in salt water is subjected to the destructive action of marine borers, and the part above the water line, and also the part which is alternately wet and dry, is subject to decay and fungous growth, dry rot, etc. Wood constantly submerged in fresh water does not suffer much from decay or insect life. Accordingly it is necessary to creosote piling to be used in fresh water only in that part from say 2 ft. below low water mark up to the top of the pile, while in the case of piling to be used in salt water the wood should be impregnated from a point say 4 ft. below the mud line to the top of the pile. Heretofore all piling used in this country, which has been creosoted at all, has been treated the full length, and on account of the fact that the small end or top of the tree (as it grows) contains a much larger percentage of sapwood than the butt, the end of the pile which goes into the ground has a much greater amount of oil, per cubic foot, than the butt, or in other words, the part of the stick requiring no preservative gets the greatest amount of preservative fluid. By the above process substantially no oil is used to impregnate that part of the wood which does not require preservation, the small amount absorbed apparently entering through small breaks or cuts accidentally made in the inner bark or skin. It is preferable to remove all the skin from that part of the length which it is desired to fully impregnate, but small pieces, say not greater than an inch wide by 5 or 6 in. long will not prevent substantially complete impregnation at these points.

ARTESIAN WELLS AND METHODS OF PUMPING THEM.*

By John D. Kilpatrick.

THERE has not hitherto been much discussion on the subject of the design, construction and operation of water supplies obtained from underground sources, particularly those from driven wells. In a great many localities the underlying formation is such that a water supply from a driven well system is impossible to obtain, but there are hundreds of situations where it is possible to obtain an artesian supply, and consideration should be given to this source, even when a filtration plant is available.

It is often stated that artesian supplies are not permanent in character. The writer disputes this point, because his experience, extending over a great many years, has led him to believe that the fault is not with the underlying water-bearing strata, but either with the construction of the wells themselves or to bad condition, due to neglect. There are many cases where the yield from the wells has steadily fallen off, and where it was possible, by developing the wells, and possibly changing the method of pumping, to bring the wells back to their original and sometimes to an even greater yield. . . . The writer refers particularly to wells driven through sand or gravel formation, where the use of a strainer is necessary. Of course, in the case of wells driven into the rock, it is only possible to obtain from any rock-hole the amount of water contained or flowing through the crevices and fissures in the rock. With sand holes, however, the yield from any well is greatly affected by methods of construction and development of the well when drilled. The writer has drilled wells that yielded 15 gallons per minute which have been brought up to over 300 gallons per minute before being put into service.

In places where there is a choice between artesian well water and a surface water supply that has to be filtered before it is possible for the water to be used, more consideration should be given to an artesian supply if the cost of the plant and the cost of water delivered to the pressure pumps is close enough to allow of debate. This opinion is based solely on the ground of the purity of the supply. . . .

The writer refers particularly to the purity of supplies from driven wells, and admits at the same time that driven well water in almost every case is harder water than that obtained from surface streams. In very few cases, however, does driven well water have to be treated either to clarify or purify it. The installation of a filter plant for the clarification of a surface supply is not the end of that problem; ceaseless vigilance in the care of the filter is the only price of safety. There are localities where it is debatable whether an artesian supply is better and more economical to operate than a supply taken from a surface stream that is to be filtered, and still other localities where there are no surface streams, and the municipality is compelled to resort to artesian wells.

The problems involved in the laying out of an artesian well system may be grouped under three heads: (1) Location of the wells; (2) methods of drilling and construction; (3) methods of pumping. No consideration will be given in this paper to the methods of pumping from the pump-house up to the storage reservoir, nor to distribution systems. Only the delivery of the water to the suction basin in connection with the main pumps will be treated.

The location of the wells depends upon the extent and surface conditions of the land available for the well field. No rules can be set down for the proper location of wells until test-wells have been drilled, unless the underlying conditions are known from wells in the vicinity. In laying out a pumping plant which will obtain the supply from wells, as a general rule it is best to drill the wells and locate the pump on the lowest possible ground available. The obvious reason for this is that in drilling the wells we penetrate to the underground stream, and the object is to have the pumped level of the water as close to the surface as possible, because, as a general rule, it is more expensive per foot of pump head to deliver the water to the surface of the ground than from the surface reservoir up to the storage tank. If the well field is to be in a well-defined valley, it is preferable to drill the wells in a line across rather than parallel with the direction of the valley, because the underground stream of water may flow in the same general direction as the surface streams. This, however, is not always the case, and before locating definitely any number of wells, it is advisable to drill test-wells at various points.

There have been cases where city councils have taken the matter into their own hands, and have drilled wells on the summit of the hill on which they have decided to place the storage reservoir under the specious reasoning that if they could get a flowing well in the valley, they should get on top of the hill a flowing well, delivering its water freely into the storage reservoir, which would save the expense of pumping. Unfortunately, water will not rise above its source, and the money spent on drilling these altitudinous wells has been thrown away. There are places where it is possible to obtain flowing wells on the summits of high hills, but they are the exceptions which merely prove the rule.

In regard to the location of underground water supplies by the use of forked sticks, magnetic balls, pendulums, indicators for radio activity and other devices, the belief in such occult means is only evidence of the survival of a superstition of the alchemist's age.

The only scientific method of determining the location for a well is to obtain all the information possible about wells in the vicinity, together with all possible data regarding the underlying geological formations. A combination of experience with the aforesaid data and information is of considerable value. In addition to this information, test-wells should be drilled to the maximum depth considered necessary, and should not be less than 6 in. in diameter, so that a pump of reasonable size can be installed, and some conclusion drawn from the yield of the "prospect well." After these "prospect wells," the number of which will depend upon the local conditions, have been drilled and tested, sufficient data is at hand upon which to estimate the total number of wells that will have to be drilled to give the required yield. Great care must be taken to obtain an accurate record of all the formations passed through, and frequent tests must be made to determine the yield at different depths.

Driven wells may be roughly divided into two classes: (1) those where the water supply is obtained from the rock, and (2) those where the water-bearing strata lie above the rock. A third class might be added, a combination of these two.

(1) Where the rock lies at a short distance below the surface of the ground, and the quality of the water

*From the "Transactions" of the American Waterworks Association.

above the rock is unsatisfactory, the pipe should be driven so as to seat firmly into the rock, shutting off the surface water. The most satisfactory method of doing this work in the case of the 8-in. finished hole in the rock is to drive a 10-in. pipe down to the rock and drill a 10-in. hole in the rock far enough to be surely into the solid rock and below the shattered and seamy top surface. An 8-in. pipe should then be lowered to seat into the bottom of the 10-in. hole in the rock, and cement grout poured in sufficiently to fill up the annular space around the 8-in. pipe in the 10-in. hole in the rock. After this is set and the 8-in. hole drilled on in the rock, the 10-in. pipe may be withdrawn for use in another well.

During the drilling of the well tests should be made by means of sand bucket or working barrel to determine the yield at different depths. A good indication of passing through water-bearing crevices is the rise or fall of the standing water in the well. If continued drilling shows the same character of rock and the standing water level remains the same, it is a good indication that there is no great change in the possible yield of the well. Another indication of very little water in a rock well is shown by a great rise in the water-level in the well when the drilling tools are lowered.

When the well has reached the required depth, a test should be made, either with the working-barrel or the air lift. If there are a number of wells to be put in, the air lift system is to be adopted for the permanent pumping plant, and the test is made with a working barrel, it is advisable to place the working barrel at a point below the surface, so as to leave sufficient depth for the submergence of the air left. For instance, if the wells are 200 ft. in depth, the writer would not recommend that the working barrel be placed more than 100 ft. below the surface of the ground and the yield determined at this point.

(2) The construction of wells in sand or gravel. In driving wells through sand or gravel it is essential that the drive pipe be of strictly wrought iron and equipped with patent recessed couplings, and care must be taken so that the ends of the pipe butt in the couplings, and that the pipe be shod on the lower or cutting edge with a steel drive shoe. The reason for the pipe butting in the couplings is to carry the effect of the blow of the tools directly through the pipe to the drive shoe, instead of having the impact come upon the threads in the couplings.

The proper strainer and the placing of it is the next point to be considered. If the strainer is to be placed at the bottom of the well, this may be done either by driving the pipe through the water-bearing strata, introducing the strainer, and packing back the drive pipe so as to uncover it, or the drive pipe may only go to the top of the water-bearing sand, and the strainer pumped or driven into proper position. In the first case the strainer may be plugged before being lowered, but chances would be taken in the ability of the well-driller to jack back the drive pipe. In the latter case difficulty is sometimes found in placing the plug securely.

In either method of construction it is essential that means be taken to prevent sand from running up alongside the strainer between the top of the strainer and the well casing. This sand is kept out either by putting in a lead packer or by continuing an extra line of pipe from the strainer up to the surface of the ground. Where the lead packer is used the strainer is lowered into the hole, and in order to withdraw it any time considerable difficulty is usually found in getting hold of it and getting it out of the well without destroying the strainer. As a general rule, it is better to lower the strainer into place by means of piping extending all the way to the surface of the ground, and then, if it is necessary at any future time to withdraw the strainer it is a comparatively simple matter to do so.

In case the water is found in three or four strata with some difference between them, it is necessary to drive the pipe to the extreme depth, and then lower the strainers into place, with the proper connecting pipes between them, and then jack out the well casing at least as far as the top of the uppermost strainer.

The operation of jacking back pipe is one involving risk on account of the pipe parting under the strain. The only precautions that can be taken are to sand pump freely and frequently when the pipe is being driven, have the strainers on the ground, and lose no time in placing the strainers before jacking back the pipe, so that the sand and gravel will have as little time as possible to pack around the drive pipe couplings. Cutting the drive pipe may be resorted to in case it is unnecessary to withdraw the lower part of the drive pipe, which may have been driven to give the necessary amount of submergence in the case of an air lift well. The necessity of insisting upon the pipe being butted in the coupling is observed more particularly when jacking operations are required, because if the pipe is not butted there is even greater danger of stripping the threads. In case a pipe parts when being jacked back, it may result in a lost well.

(3) Under the third heading, where water may be obtained from the formations above the rock and from crevices in the rock, the construction of the well differs in no way from those referred to in the first two actions.

The pumping of driven wells may be done either by suction, deep well pumps, rotary or screw pumps, or the air lift. In case the water rises high enough in the wells to be pumped by suction, it is unnecessary to speak of this, other than to say that method of pumping is familiar to anyone who has ever done any hydraulic work. The only prime necessity for a successful suction plant is tight suction lines and good foot valves.

In regard to deep well pumps, these may be used when only limited supplies are required and where the water does not rise in the well high enough to make an air lift economical. The question of whether to use single-acting well heads with single-acting barrel, or single rods with double-acting barrel or double rod heads with double plungers depends entirely upon circumstances. Except under extreme conditions, the writer does not believe it advisable to use deep well pumps where there are more than two wells on account of the spacing between the wells, and the necessity of building separate pump-houses over each well, and for other obvious reasons of economical pumping.

For any number of wells scattered, as they usually must be, over quite an area, the air lift system has proven itself to be the most available method for delivering a large supply of water on the surface of the ground. It is not advisable, however, to use the air lift, except under extraordinary conditions, for delivering the water higher than to the level of the ground. At best, the air lift is an expensive way of pumping, but where the water is obtained in large volume below suction limits it would seem to be the only possible method. The air lift system has the great advantage of not having any moving parts in the well, and there is absolutely nothing to get out of order in the air lift itself. The moving parts of the system are all in the air compressor in the engine-room, under the eye of the engineer. The engine-room may be located at the most convenient point, taking into consideration the supply of fuel, and the furthest well may be a mile or more away from the air compressor. If the air lines are correctly designed and properly buried, there need be but little loss of pressure. The main fault that the writer has found with a great many air lifts throughout the country has been in the air lift piping in the wells.

It is a familiar fact that a prime consideration for an economical air lift is that there should be 60 per cent. submergence. For instance, in a 100-ft. well there should be at least 60 ft. of water when the well is delivering its yield; but the principal trouble is found in the design of the air and water piping, so that the compressed air is delivered at the bottom of the uptake water pipe with as little loss by friction as possible, and that the uptake water pipe is designed so as to deliver the water, without being so large as to have an extreme amount of slip or so small as to develop excessive friction. Evidence of faulty design is shown by the discharge being in alternate pistons of water and air.

In a properly designed air lift system the water should be discharged in a practically uniform stream, with very little surging. A great many foot pieces have been designed which have for their object the spraying of air at the bottom of the uptake water pipe, so as to introduce the air into the water in streams of very fine bubbles. It was discovered early in the development of the air lift that the finer the spray could be made, other things being equal, the more efficient the air lift became; but no foot piece that was ever designed can work efficiently if the air pipe leading down to it, and the uptake water pipe from it, are not of such sizes as are suitable for the particular problem involved in that well. The great object to be accomplished by any foot piece is to offer as free and clear a water passage as is possible, so that there will be no eddies formed in the water column which cut down the velocity and impair the efficiency. Above the foot piece the mixture of air and water should have a pipe with as smooth surfaces as it is possible to obtain.

The three methods of piping wells may be termed: (1) The outside air pipe; (2) the inside air pipe; (3) the annular system. In the first case the air passes down outside the water pipe and into some type of foot piece or openings at the bottom with the water passing up through the inside of the water pipe. In the interior pipe system the air passes down the inside air pipe to a foot piece or nozzle, and the mixture of air and water is blown upward between the outside of the air pipe and the inside of the water pipe. In the annular pipe system one pipe is within the other, the space between the two pipes being the downtake air column, and the interior of the inside pipe being the uptake water discharge pipe. It is impossible without an exact knowledge of the conditions to determine which of these three systems it is best to use, assuming that the proper areas are available in each case for the amount of air necessary and the amount of water required to be lifted.

There are cases where the diameter of the well is so small and the amount of water to be delivered so large that there would not be room in the well for the outside air pipe, in addition to the uptake water pipe. In this case it might be better to use the well casing as the downtake air pipe, and only provide an uptake water pipe. This arrangement, however, is sometimes impossible on account of air leakage through the joints of the well casing, which were opened up when the well pipe was driven. It is very hard to generalize and lay down rules for piping up wells on account of the different yields, depths and submergence of wells, and this is sometimes still further complicated by the number of wells that have to be pumped.

With a great number of wells to be pumped from the same air compressor, very delicate adjustments are required so that all the wells may be started at the same time. The writer believes that it is preferable to use a single air line, with outlets to each of the wells, rather than separate air lines from the pump-house to each of the wells, both as a matter of economy of installation and economy in the use of air. The plea sometimes made that independent air lines to each well allow of adjustment within the engine-room is valid, because the place to adjust the well is at the well itself, and if the plant is properly designed originally, it should not be necessary to adjust the wells, except at considerable intervals of time, and then only because of the increased requirements made on the plant or fluctuations in the wells themselves.

Another rule, the observance of which should be insisted upon, is that the air lift should only be used to deliver the water high enough above the ground to allow of a flow to a surface suction basin close by the force pumps.

The writer has not referred to air pressure machinery, but considerable economies can be made in this part of an air lift system by installing compound and condensing machines and by two-stage air ends. The great loss of economy in the ordinary air lift system, however, is not in the engine-room, but in the air lift pump in the wells. Insufficient submergence, caused by pumping too great a quantity from the well and thereby lowering the head to a point where the increase in the amount of air necessary is out of all proportion to the quantity of water obtained, or by the use of piping either too large or too small. No foot piece ever designed will do the impossible, but a properly designed foot piece in connection with correctly designed air and water pipes will make the air lift an important factor in the pumping of deep wells, and a properly designed air lift system in connection with wells that have a good flow of water will result in pumping costs which will compare favorably with the cost of some surface water supplies that have to be filtered.

THE PIONEER OF HEAVY ELECTRIC TRACTION.

The first electric locomotive used for hauling freight was, according to Aera, the product of the ingenuity of Chas. J. Von Depoele. This locomotive was built by the Pullman Company and was put in operation in 1888. After about fifteen months' service it was scrapped, not because it was worn out or had outlived its usefulness as a freight carrier but because the owners found that passenger traffic paid higher dividends than heavy traffic. The motor developed 75 horse-power and was capable of hauling a maximum load of 35 tons.

As a preventive of disintegration in a large rock cut on the New York Central, the cement gun has been successfully used to apply a coating covering the rock and filling the external crevices so as to prevent further frost action.

The Department of Public Highways, Province of Ontario, has issued another edition, revised to date, of its rules for the guidance of road superintendents and engineers in county road construction and repair. These rules were originally published in 1911.

HIGHWAY BRIDGE DEVELOPMENT IN ONTARIO.*

By Geo. Hogarth, A.M.Can.Soc.C.E.

We have to-day, sections in the south of the province that have been settled for over a hundred years; and in the north, there are vast areas where the axe of the first settler is only now being heard. Our bridge construction, therefore, varies from the most primitive types of timber construction suitable for the lightest of traffic to the more enduring structures of concrete and steel, which are capable of safely sustaining the weight of a twenty-ton road roller.

Our rivers of the north are usually broad and deep with nothing more secure than a shifting, slippery clay bank upon which to build abutments or piers. The crossing of such rivers is an expensive undertaking since the river bottom is frequently soft and very liable to be deeply scoured if the current is in any way deflected by a pier. In Ontario, hundreds of bridges are required each year, and those large structures which are more expensive and serve only a small population must frequently give way to less costly, smaller structures or ferries, which furnish communication till bridges are warranted. In deciding on the type of bridge to build, consideration must be given to the lumberman who is bringing sawlogs down the river, and piers must be located or omitted with a view to avoiding log jams. The safe location of the piers usually governs the length of bridge span to use, since the crowding of logs cannot always be prevented, and a heavy jam will often pull timber piers clean out of the river, piles and all. There is also the ice to contend with, and it works almost unceasingly to destroy any timber structure with which it comes in contact. Late in the fall, when the water is low in the river, the ice forms and sticks solidly to the piles or cribs for a depth of probably three to four feet. Should a sudden thaw come in February, the water lifts the ice and gives the piles a heave that throws the entire structure out of grade. For these reasons, it has been found advisable to bridge the rivers with one span wherever possible, and to place the abutments or piers out on the banks of the stream. The placing of piers in the river channel is usually an expensive piece of work, and the maintenance money that must be spent to protect them from logs and ice in the spring of the year is frequently considerable. The use of long-span bridges is therefore an economy.

On the smaller creeks and rivers, the timber queenpost bridge is still built and is supported by pile piers or timber rock-filled cribs; but where the bridge must be 60 feet or over in length, a steel span is the best and cheapest type of structure that can be built. The cost of labor in remote sections of the north is sometimes out of all proportion to the work done, and these types of construction have been developed from actual experience as being the most economical under present conditions. With such a type of bridge, the building of the pile piers requires comparatively little work, and four or five men and a team will finish the timberwork, erect the steel span and lay the floor of an ordinary structure in about three weeks' time. Many such bridges are built in locations which are 25 and 30 miles from a railroad, and local men accustomed to the country must be employed, since the ordinary discouragements of life on that class of work drive the new comer out of the business.

*Paper read before the 3rd Canadian and International Road Congress, Montreal, March 6-10, 1916.

The highway bridges built in the settled districts and counties of older Ontario are of a more advanced type of construction. They must be capable of carrying heavier loads and be built so as to withstand the wear and tear of greater traffic. Since good sand, gravel and crushed stone are easily obtained and cement is cheap, it is economical to build the structures of concrete. For the longer spans, where concrete is not as serviceable, the steel bridge is used, and it is customarily supported on concrete abutments and provided with a concrete floor.

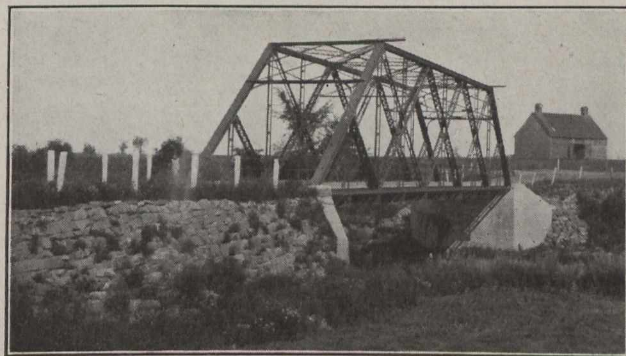


Fig. 1.—Steel Bridge Over Carter River at Charlebois.

For short spans, the concrete beam bridge is a very desirable structure, and for spans of medium length, where local conditions admit of its use, the concrete arch is of pleasing appearance. Such work, when well designed and properly built, is very durable, but great attention must be given to the foundations and to the surrounding conditions in order that damage to the structure may be prevented. Concrete is easily adapted to almost any foundation, but it is not in the best interests of such construction to use it in important locations where settlement of the piers is to be anticipated, or where the river channel will be cramped, due to a low bridge being required. The safety and enduring qualities of a concrete structure de-

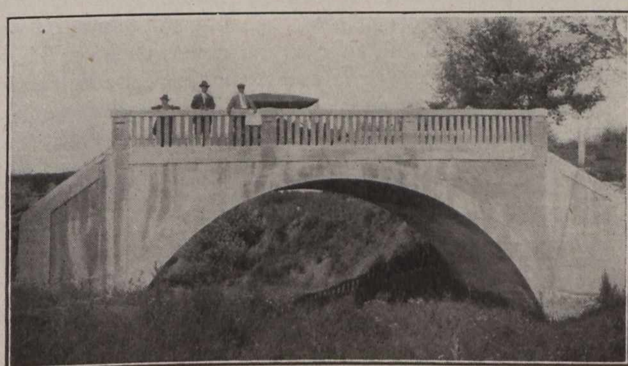


Fig. 2.—Thirty-foot Concrete Arch Bridge in Russell Township.

pend to a great extent on the stability of the foundations. The slightest movement of the footings will result in cracks opening in different parts of the structure, and while such cracks may not seriously affect the strength or safety of the bridge, they are unsightly and indicate an undesirable condition of affairs. Concrete is a splendid building material and will give good results even with very indifferent workmanship. It is particularly well adapted to certain locations and designs, but if used indiscriminately failures are bound to occur. In the case of small concrete bridges, the placing of the footings at a sufficient depth below the ground or water surface is frequently disregarded. As a result, the rush of water during a freshet undermines the foundations and the entire structure may be lost. It is good practice to carry all footings down to a depth of at least four feet below low-water level since at that depth the foundation will be safe from frost as well as from the scouring action of the water. There are so many vital considerations entering into the design of a highway bridge that the selection of the type and nature of the bridge should be left entirely to the engineer and his decision should be final.

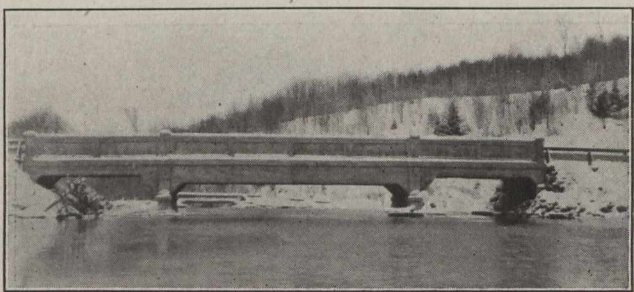


Fig. 3.—Concrete Beam Bridge Near Kearney.

Our highway bridges are now designed to carry a concrete floor and a 15 or 20-ton road roller, or a live load of 100 pounds to the square foot of floor surface. The structure which is built to carry such loading is of good proportions with fairly stout members. In the past, insufficient quantities of metal were used in many bridges and they were built so light as to be unable to ascertain for many years the wear and tear to which they were subjected by the traffic and the elements.

The tendency of the times indicates that a 20-ton road roller will be the maximum load for bridges for some time to come. In some localities it is proposed to limit the weight of road rollers and auto trucks allowed to pass over highways and to make the maximum permissible weight of such machines 10 or 12 tons. Legislation along such lines appears to be advisable since with an unrestricted weight of auto trucks we would soon see excessively heavy vehicles doing considerable damage to highway bridges. Some timber and concrete floors have already received severe treatment and have been partly destroyed by the heavy rear axle weight of loaded trucks, and unless steps are taken to curtail such weight the damage will greatly increase. It is advisable, therefore, to enact laws which will set a limit on these heavy loads so that the highway bridges can be built in the security of knowing the heaviest load they will be called upon to carry. Otherwise, great confusion may result, and a condition might arise where all our bridges would be too weak the moment an auto truck manufacturer increased the capacity and weight of his product. The establishing of a definite maximum load gives the auto builder and the bridge engineer a basis upon which to develop and improve all designs.

While we now build what is believed to be a fairly stout bridge, the required minimum thickness of metal of 5/16 inch causes nearly all highway bridge work to be known as tinwork in the shop where it is fabricated. If durable structures are to be constructed, no skimping or trimming out of metal should be allowed. It would appear to be a step in the right direction if no metal less than 3/8 inch thick was permitted to go into a highway bridge. Steel highway bridges are still built too light and flimsy to give a long length of life. Many steel bridges in use 25 years urgently require renewal to-day because of the serious rusting of the thin material, and if our work is to be enduring and have a fair length of life, it is absolutely essential that a sufficient quantity of metal be used in the new bridge.

When a bridge is to be built, accurate information is required respecting the width of the river, the depth of the water, the height of high and low water, the navigation or log driving to be provided for, the manner in which the ice goes out in the spring, the quantity of driftwood brought down by the freshet, the nature of the banks of the river, the character of the foundations that will be required, the local material available for concrete or timberwork, and the distance to the nearest railway station. This information is necessary, be the bridge small or large, and in addition the judgment of the engineer comes into play when the question is put as to what structure is best adapted to the site. Many instances could be cited where, owing to incomplete information, a bridge pier was placed in the middle of a river. The amount of money required to protect and maintain that pier, together with its first cost, would have paid for a steel bridge long enough to completely span the river. In some cases, bridges of insufficient length have been placed at crossings of wide rivers and as a result they have been swept away on the crest of the first serious flood. The position of the banks of a river is very significant. They are standing evidence that, at one time or another, the river possessed sufficient force and power to sweep away everything between those banks, and a structure which cramps that wide waterway is putting up a losing fight with nature. The pages of our engineering journals continually record the washing away of bridges, and the lesson frequently placed before us is that the waterways provided at bridges should be of a sufficient size to pass the floods. The creek of to-day may be the roaring torrent of to-morrow, and provision should be made for that excessive rush of water. It is only natural to build bridges as small as possible and to construct them with the least expenditure of money, but in building a bridge the first consideration should be the safety of the completed structure and the size of the waterway to be provided must govern the design.

The nature of the foundation on which the abutment of a bridge is to be placed deserves mention. An engineer

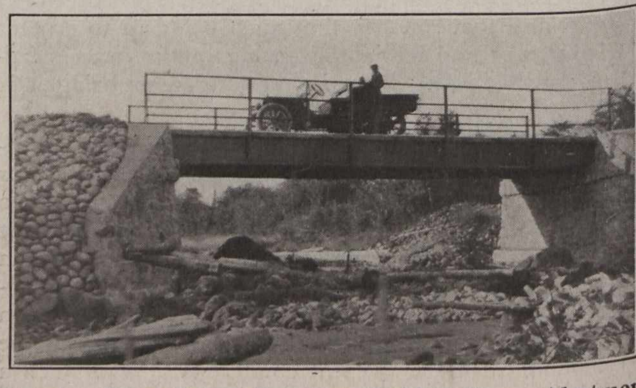


Fig. 4.—Steel Beam Span on Rubble Stone Abutments Near Sault Ste. Marie.

is called upon to construct bridge footings in every kind of location from one that is a bottomless bog to one that is splendid solid rock. There are between those extremes a number of different classes of material, all of which require close attention in order that a secure footing may be obtained. A solid rock foundation is ready for the concrete as soon as all the loose and decayed rock has been blasted and cleaned away. It is well to have the rock footing fairly level and yet rough enough to give the concrete a good bond to prevent any possibility of sliding.

With a foundation of gravel, boulders, or hardpan, the concrete of the footing may be deposited when the excavation reaches a depth of four or five feet, since at that depth scouring of the material cannot occur and frost will have no effect.

In some locations where an exceptionally soft foundation occurs, it is advisable to divert the road to a better crossing, where a more secure bottom can be obtained. With a highway bridge, however, there is a lack of vibration and impact and the loading is comparatively light, so that in almost all soft locations a good pile footing will be found to be all that is required to safely carry the abutment.

For short bridges up to, say, 40 or 45 feet in length, and where the bottom is soft, a steel beam bridge gives a very satisfactory structure. The small cost of the entire work does not justify expensive pile foundations, and a mat composed of long timbers may be laid in the bed of the river so that each timber extends under both abutments.

A slight settlement is to be expected with such a structure, but no harm or damage to the bridge will occur.

The maintaining of the many steel bridges now on the highways is a work requiring considerable experience and attention. A bridge is like any other structure built by man—it is not everlasting. In the case of steel structures, it has practically been a custom to neglect them and they are seldom painted. This neglect hastens the rusting and decay of the metal, and the day soon comes when another bridge is necessary. It is frequently a difficult matter to have councils appropriate money for painting bridges when they have seen indifferent and expensive work done on bridges in their own or in an adjoining municipality. There is no doubt whatever that money spent for painting is real economy, and there is no defence that can be offered for allowing a bridge to go to ruin. If a structure is painted every four years, it will take five complete paintings to protect it for 20 or 25 years, and at the end of that time it should be in a good state of preservation.

Fig. 5.—Concrete Beam Bridge in Cumberland County.

It could then be removed to a highway having lighter traffic and would probably be of good service in that new location for a number of years. Experience with bridges that have been uncared for for 25 years, indicates that they are just about ready for the scrap heap; whereas, proper painting, carried out at comparatively small cost, would have rendered them still useful for an indefinite period.

The practical test of observing the bridge during the passage of a heavy load may result in the discovery that the various parts appear to be loose and that the entire structure appears to be working or moving. If there are a number of adjustable members in the trusses and lower laterals, it is probable that the tightening of such while no load is on the structure will cure any apparent looseness, while if the bridge is fully riveted it is desirable that close attention be given the various joints to see that rivets are still tight. If a number of loose rivets are found, it is best to cut them out and re-drive so as to produce a tight joint.

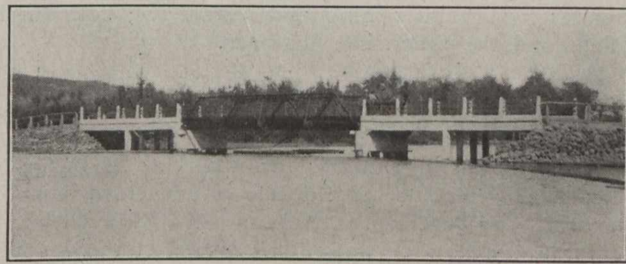


Fig. 6.—Steel Bridge with Reinforced Concrete Trestle Approaches, all Supported on Concrete Piles.

In Ontario, we have many concrete bridges, and where such structures were originally well built there are no maintenance charges and little or no inspection required. The first concrete arch bridge built by the Department of Public Works was constructed in 1907. It is founded on solid rock, and to date not one cent has been expended for maintenance.

In conclusion, it may be said that the highway bridge is to-day in an important stage of development. The knowledge gained in using the various materials of construction is tending to modify and improve the design and general appearance of such structures, and a more artistic type is being aimed at. With an established system of loads for all structures, and a greater public demand for permanency in construction, a considerable improvement in the character and type of bridges is to be expected.

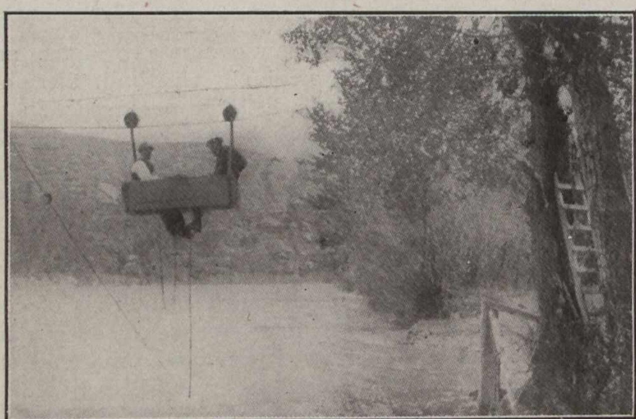
An endless chain operated by an electric motor is used in loading ties for shipment from a Texas lumber camp. Two men place the ties on the chain at one end and two men remove them at the other end and place them on the car.

The large amount of zinc required for war purposes, and the resulting enormously increased demand for the metal, lend special interest to an article in the current number of the Bulletin of the Imperial Institute, on "The Occurrence and Utilization of Zinc Ores." The chief zinc minerals are described, and a brief account given of the more important occurrences in the United Kingdom, the Colonies and India. Zinc ores have been mined in many parts of the United Kingdom, notably in Cumberland, Northumberland, Durham, Derbyshire, Shropshire and the Isle of Man, but a large proportion of the production has for several years past been shipped to the Continent for smelting. By far the most important zinc deposits in the British Empire are those of Broken Hill Mines, New South Wales, the output of which alone is sufficient to supply the entire demands of the United Kingdom for metallic zinc. The Broken Hill ore before the war went mainly to Germany for smelting, but the Australian Government has adopted measures which will prevent this in the future. Zinc is also found in South Australia, Queensland, Tasmania, New Zealand and Newfoundland. Canada contains a number of workable zinc deposits, particularly in British Columbia, and there is every prospect of Burma becoming an important producer. In Africa there are zinc deposits in Egypt, Nigeria, Rhodesia, and the Transvaal, as to which more information is needed.

FIELD AND OFFICE METHODS EMPLOYED BY THE HYDROMETRIC SURVEY OF CANADA.

In the recent report of Progress of Stream Measurements in Canada, which has been prepared under the direction of F. H. Peters, M.Can.Soc.C.E., by the chief hydrometric engineer, P. M. Sauder, M.Can.Soc.C.E., and assisted by G. H. Whyte and G. R. Elliott, A.M.Can.Soc.C.E., some information on stream-gauging is given. The theory involved is not new, but a great wealth of detail as to the application of it makes it very interesting and instructive, both to the student and the practising engineer.

Stream Measurements.—There are three distinct methods of determining the surface flow of streams: (1) by measurements of slope and cross-section and the use of Chezy's and Kutter's formulæ; (2) by means of weirs, which include any device or structure that by measuring the depth on a crest or sill of known length and form, the flow of water may be determined; (3) by measuring the velocity of the current and the cross-section. The third method is the one most commonly used by this survey. The second is used when the flow



Gauging Station on Old Man River, near Cowley, Alta.

is too small to be accurately determined by the third, while the first is only used in making estimates of the discharge of a stream when the only data available are the cross-section and slope. The slope method of determining discharge will not be discussed, as it is only approximate. The weir method is applied for small streams. Few permanent weirs have been installed by the survey. Many weir measurements have been made by means of temporary weirs, which consist of a wooden base of 2-inch plank with a rectangular notch of ⅜-inch steel bolted to it. The edge of the steel is bevelled. Care must be exercised in erecting the weir properly and in the choosing of a good location for it. The depth of water on the crest should not exceed one-third the length of the weir. The approach channel should be several times as wide as the opening, and the depth of water in the pond should be twice that over the crest so as to eliminate velocity of approach and cross-currents.

The dam in which the weir is set should be at right angles to the direction of flow. Sods are generally used in its construction, and precautions must be taken to prevent undermining. When the bay has filled up the head of water is determined by taking levels at the crest of the weir and at the level of the bay, from 4 to 10 ft. upstream; the difference of these gives the head operating at the weir.

After determining the head the discharge is computed by using the following formula, which is a modification of Francis' formula for rectangular, sharp-crested weirs: $\phi = 3.33 (L - .2H) H^{3/2}$, in which ϕ = discharge in sec. ft.; L = length of crest in ft., H = head in ft.

Measurements by means of temporary weirs should be made some distance above or below the gauge. If they are made close to a gauge, the gauge must be read before the weir is placed in the stream, and the pond must be allowed to run off after the weir is removed before the gauge is re-read.

Where permanent weirs are installed, the gauge height observed is that of an auxiliary gauge above the weir, which is kept so that the head of the weir can be read direct. The weir is not usually placed so that it will interfere with the regular station, so that if at any time the weir is destroyed the regular gauge can be read during the period that the weir is out of order.

The velocity method of determining the discharge of a stream is the most accurate. There are two methods of determining the velocity of flow of a stream, namely, direct and indirect. In the direct method, by which the velocity is determined by means of floats, the liability of error is large and the results far from satisfactory. This method is seldom used except for very rough estimates, or when a current meter cannot be used.

The indirect or current meter method is the most reliable and most widely used method of determining the velocity of the flow of a stream. The meter used by this survey is the Price Patent, manufactured by W. & L. E. Gurley, Tróy, N.Y. It consists of six cups attached to a vertical shaft, which revolves on a conical, hardened steel point when immersed in moving water. The number of revolutions is indicated electrically. The rating or relation between the velocity of the moving water and the revolutions of the wheel is determined for each meter by drawing it through still water for a given distance at different speeds and noting the number of revolutions for each run. From this data a rating table is prepared which gives the velocity per second of moving water for any number of revolutions in a given time interval.

In making a measurement with a current meter, a number of points, called measuring points, are measured off above and in the plane of the measuring section, at which observations of depth and velocity are taken. These points are spaced equally for those parts of the section where the flow is uniform and smooth, but should be spaced unequally for other parts according to the discretion and judgment of the engineer. In general, the points should not be spaced farther apart than 5 per cent. of the distance between piers, nor farther apart than the approximate mean depth of the section at the time of measurement.

The measuring points divide the total cross-section into elementary strips, at each end of which observations of depth and velocity are made. The discharge of any elementary strip is the product of the average of the depths at the ends, the width of the strip, and the average of the mean velocities at the two ends of the strip. The sum of the discharges of the elementary strips is the total discharge of the stream.

There are a number of different methods of determining the mean velocity at the ends of these strips, or, as it is commonly called, the mean velocity in a vertical, namely, multiple-point, single-point, and integration. These three principal multiple-point methods in general use are the vertical velocity-curve, three-point and two-point method.

In the vertical velocity-curve method the centre of the meter is held as close to the surface of the water as possible, being careful to keep it out of reach of all surface disturbances, and then at a number of different depths throughout the vertical. The velocity at each position of the meter is recorded. These observations are then plotted with velocities in feet per second as abscissae and their corresponding depths in feet as ordinates, and a mean curve is drawn through the points. The mean velocity for the vertical is obtained by dividing the area bounded by the curve and its axis by the depth. In the absence of a planimeter for measuring the area, the depth is divided into 5 to 10 equal parts, and the velocities of the centre ordinates of these parts are noted. The mean of these velocities will very closely approximate the mean in the vertical.

The vertical velocity curve is useful in studying the manner in which velocities occur in a vertical. From a study of a number of these curves the other shorter methods of determining mean velocity are deduced. On account of the length of time taken to complete a measurement this method is not used in general routine measurements, except during the winter, for a change of stage is almost sure to occur during a measurement on a large stream, which counterbalances the increased accuracy. For this reason its use is limited to the determination of the coefficient to be used in the reduction of values obtained by other methods of measuring velocity to the true value, to the measurements of velocities under new and unusual conditions of flow, and for measurements under ice.

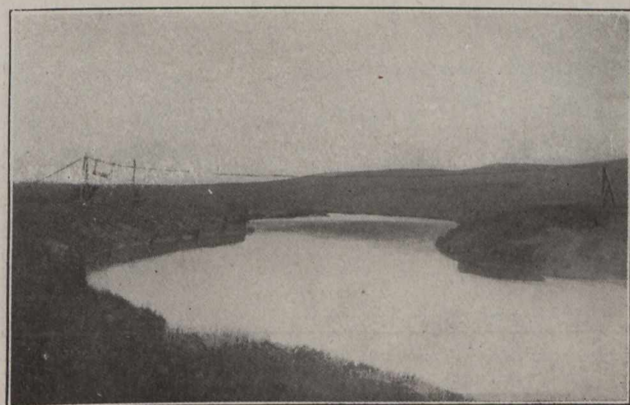
The three-point method is one of the short methods of obtaining the mean velocity in the vertical, and, under some conditions, gives the most accurate results next to the vertical velocity-curve method. It has been used almost exclusively by this survey in past years, during the open-water period, but recently has been superseded by the two-point method, which, under most conditions, gives more accurate results. In the three-point method the current-meter is held at 0.2, 0.6, and 0.8 depth. The mean is then obtained by dividing by 4 the sum of the velocities at 0.2 and 0.8 depth, plus twice the velocity at 0.6 depth.

In studying the vertical curves made at a number of different points and under varied conditions, it has been found that the mean of the velocities occurring at 0.2 and 0.8 depth gives very nearly the mean velocity in the vertical. Use is made of this fact in the two-point method of determining mean velocity, the meter being held at 0.2 and 0.8 depth in the vertical. This method has been found more accurate than the single-point method, and the time required for a metering is not very much greater. This method has been found to give, also, a very close approximate to the mean velocity in measurements of ice-covered streams, although these flow under very different conditions from those of open water.

The single-point method is based on the results of experiments, which have established the point of mean velocity in a vertical at 0.6 of the depth. Therefore, the error resulting from the use of the 0.6 depth as the depth of mean velocity is very small, though in some few cases a study of the vertical velocity-curve will show the need of a coefficient to reduce the observed velocities to the mean. The variation of the coefficient from unity in individual cases is, however, greater than in the two or three-point method, and the general results are not as satisfactory. For that reason this method is not employed very extensively by the survey.

In the other principal single-point method the meter is held near the surface, at from 0.5 to 1 foot below the surface, care being taken to sink the instrument below the influence of wind or waves. The resulting velocities must be multiplied by a coefficient to reduce them to mean velocities. This coefficient, as found by a large number of experiments, varies from 0.78 to 0.98, depending upon the depth and speed of the stream. The deeper the stream and the greater the velocity, the larger the coefficient. In flood work coefficients varying from 0.90 to 0.95 should be used. This method is only used when the current is too strong to permit the sinking of the meter to any great depth below the surface of the water. It is often employed at times of flood, or when a stream is carrying a lot of driftwood or ice.

The integration method of determining the mean velocity in a vertical consists in moving the meter at a slow, uniform speed from the bed of the stream to the surface and return in a vertical direction, the time and revolutions being observed. In travelling through all parts of the vertical the meter is acted upon by each and every thread of velocity from the bed to the surface of



Gauging Station on the North Branch of Milk River.

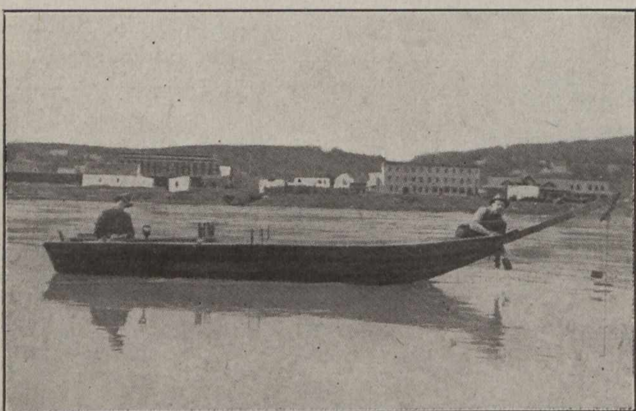
the stream, and the resulting observations determine the mean in that vertical.

This method is very useful in checking the results of other methods. It is, however, seldom used by this survey, as the Price meter is not suited to observations by this method, since the vertical motion of the meter causes the wheel to revolve.

Gauging Stations.—The location of gauging stations is very important, and the choosing of a suitable place is more difficult than one would think. Not only must the water be moving in nearly straight lines over a solid bed and between well-defined banks, but the place must be accessible at moderate cost, and there must be living near it a competent person who can be engaged to serve as observer. Permanent gauging stations should only be selected after a very thorough reconnaissance. In the irrigation districts and in more thickly populated districts there is more or less diversion of water. This is apt to complicate matters for the hydrometric engineer, for a gauging station above all works may not include all the tributaries of the stream, and it is often necessary to establish gauging stations at several points along the streams, and on tributaries, canals, and pipe lines in order to obtain complete information regarding the water supply in a particular stream.

There are three classes of gauging stations, namely, wading, bridge and cable stations. The wading station can, of course, only be used in the case of small streams having a maximum depth at its highest stage of three feet or less. The equipment for a wading station is small, consisting usually of a plain staff gauge, graduated to feet and hundredths, and fixed vertically to one of the banks of the stream. For convenience a measuring line, usually a wire with tags, may be fixed permanently at this section. When taking the reading the engineer should stand below and to one side of the meter so as not to cause eddies in the water.

Bridge stations, because of their permanency and the freedom of movement allowed the engineer, are much preferred. Very often, however, more particularly in swift currents, the piers materially affect the accuracy of the results. When the gauge cannot be attached to a pier, it is often attached horizontally to the guard-rail or floor of the bridge, and the height of the stream is found by lowering a weight by a chain over a pulley. It is indicated by a marker on the chain. Distances of three, five or ten feet, according to the size of the stream, are



Boat used for making Discharge Measurements of Athabaska River at Athabaska, Alta.

marked on the lower chord of the down-stream side of the bridge to serve as a measuring line.

Frequently it is impossible to establish a permanent gauging station at a bridge. In that case the wire cable of a ferry can be utilized, or, if that is not available, a permanent wire cable is stretched across the river. For spans of average length a galvanized wire cable ¾-inch in diameter is safe. It is supported at each bank by means of high struts or by passing it through the crotch of a tree. The cable is run into the ground and anchored securely to a "dead man," buried at least six feet below the surface, or, if convenient, it is anchored to the lower part of the trunk of a tree. A turnbuckle is inserted in the cable between the strut and anchorage to permit tightening the cable when it begins to sag. A permanent measuring line, usually a wire, with tags 5 or 10 feet apart, is stretched across the stream just above the cable. A cage large enough to carry two men and instruments is constructed and suspended from the cable by means of cast-iron pulleys. The cage is moved from point to point by hand. A stay-line, usually quarter-inch guy wire, is stretched across the stream about 30 to 40 feet upstream from the cable, and securely fastened. By passing a sash-cord through a pulley hung on this stay-line the current meter is prevented from being carried downstream. This type of station has the advantage that it can usually be located at the most desirable point on the stream and is free of piers and other obstructions.

Owing to friction in the current meter the lowest velocity at which readings can be depended upon has been found from experiment to be 0.5 ft. per sec.

Office Computations.—When a series of discharge measurements has been made at a gauging station a rating curve is constructed for that station, showing graphically the discharge corresponding to any stage of the stream within the limits covered by the gaugings. This curve, as it is usually drawn, has as abscissae the discharges in second-feet, and as ordinates the corresponding gauge heights at which the discharges were made. A smooth curve is drawn through the resulting set of points, and from this curve the discharges at any stage within the limits of the curve are taken. Some measurements may be more reliable than others, owing to more or less favorable conditions at different times of gauging, or to other causes. In order to obtain the weight of the different measurements, curves with area and mean velocity, as abscissae, and gauge heights as ordinates, are also drawn. From a study of these curves any discrepancies in a measurement, either in its area or mean velocity, may be detected. A station rating table is prepared after the rating curve is constructed which gives the discharge at any stage of the stream within the limits of the daily gauge height observations on record. From this rating table the daily discharges corresponding to the daily gauge heights are read and tabulated. The rating table is constructed for tenths, half-tenths, or hundredths of feet, according to the readings of the gauge to which it is to be applied. The discharges for this table are read directly from the rating curve and are then adjusted so that the differences for successive stages shall be either constant or gradually increasing, but never decreasing, unless the station is affected by backwater.

The rating table being made to cover the range of daily gauge height observations, the next procedure in the computations is to make out a table of daily discharges from this rating table. The daily gauge heights are copied as they were sent in by the observer, and opposite each the corresponding discharge is filled in from the rating table. The monthly discharge is found by totalling the daily discharges for the month in question, and the monthly mean is obtained by dividing this total by the number of days in the month.

The run-off is computed with two different sets of units, depending upon the kind of work for which the data is intended, as follows:—

1. Run-off in inches is the depth to which a plane surface equal in extent to the drainage area would be covered if all the water flowing from it in a given time were conserved and uniformly distributed thereon; it is used for comparing run-off with rainfall, which is usually expressed in depth in inches. The monthly mean run-off in second-feet is divided by the area of the drainage basin in square miles to find the monthly mean run-off per square mile. This result, reduced to run-off in depth in inches for the monthly period, is in the form required.

2. The run-off in acre-feet is the form of most use in connection with storage. An acre-foot is equivalent to 43,560 cubic feet, and is the quantity of water required to cover an acre to the depth of one foot. The monthly mean run-off in second-feet is used for the computation of run-off in acre-feet. The monthly mean is reduced to cubic feet per month, and this quantity, divided by 43,560, gives the run-off in acre-feet.

The run-off of the stream being computed both in depth in inches and in acre-feet for each month, the run-off for the period during which observations of run-off were made is found by the summation of the amounts of run-off for the several months making up this period.

Winter Records.—Perhaps the greatest difficulties in stream measurements are met with in the early part of the winter, just as the streams are commencing to freeze up. Especially is this true in the swift-running streams in or near the mountains. Needle and anchor ice often form in large quantities in rapids, and, flowing in masses with the water, make gaugings very difficult and unreliable. Even after a permanent ice cover is obtained at the gauging station this ice will, in some cases, obstruct the channel below the station and cause "backwater."

A further difficulty is that the surface ice usually forms along the edges of the stream for some time before forming in the centre of the channel. At first this may be broken away if the stream is small and open-water measurements made, but later it is necessary to take some observations through holes in the ice along the edge. As the streams get farther away from the mountains their velocity decreases, and fewer rapids occur along their course. There is then less trouble with needle and anchor ice, and a permanent ice cover forms much more quickly.

It is often necessary to choose a new section for winter observations. This should be done before freeze-up, for then the width, depth, uniformity of flow and conditions above and below can be easily noted. The most suitable stations for winter measurements are those which have a long stretch of very smooth, sluggish water above and a rapid fall below.

In winter as in summer, the daily discharges of a stream are computed from frequent discharge measurements and daily gauge height observations. The discharge measurements are made through holes in the ice from five to ten, or even twenty feet apart, depending upon the size of the stream, and large enough to allow the current meter to pass though freely. The gaugings are made in the same manner as at open sections except that the depth of the stream is taken as the distance from the bottom of the ice to the bed of the stream. The soundings, however, are always referred to the surface of the water in the holes, the distance from the surface of the water to the bottom of the ice being measured and subtracted from the soundings to obtain the depth.

The vertical velocity-curve method is usually used for the determination of the mean velocity in the vertical. A curve is plotted for each vertical, and the mean velocity is determined in the usual manner. These curves vary greatly as to form for different kinds and conditions of channel.

It is found that when all the holes are opened on a small, swift stream there are sometimes vertical pulsations of the water in the holes, which affect the velocity readings. This can usually be avoided by only opening one hole at a time, and filling it in again with ice and snow as soon as the observation is finished. It can also be overcome by inserting a thin sheet of galvanized tin or iron at the bottom of the hole after the meter has been lowered into the water. The meter should always be held near the upstream side of the hole.

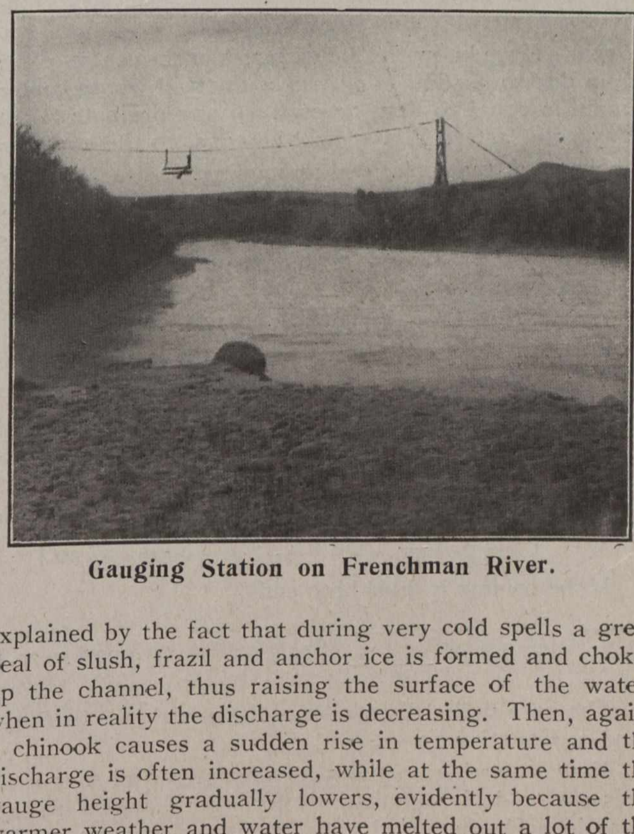
In using the meter care must be taken to keep it under the water as much as possible to prevent ice from forming around the bearings. It is a good plan to clean and oil the meter indoors before starting out to make a gauging.

Gauge Observations.—The gauge is usually read once a day, the observer cutting a hole in the ice and noting the elevation of the water and gauging the thickness of the ice by means of an L-shaped ice-gauge. Notes are taken as to needle ice, slush, snow, ice-jams, and any sudden temperature changes.

Any form of gauge may be used, but the chain-gauge is the most satisfactory, as the staff-gauge, being frozen to the ice, heaves with it, and also in cutting away the ice from around it the figures are effaced. The automatic gauge gives trouble with the well freezing over.

While the run-off, particularly during the winter months, does not vary directly in accordance with the precipitation, the rate at which it reaches the streams is, of course, dependent almost entirely upon the climatic conditions.

There is, therefore, very little surface run-off, and ern Canada which make it exceptionally difficult to make estimates of the daily discharge during the winter. The gauge height in many cases fluctuates very much, and often sudden rises or drops occur. These rises are often



Gauging Station on Frenchman River.

explained by the fact that during very cold spells a great deal of slush, frazil and anchor ice is formed and chokes up the channel, thus raising the surface of the water, when in reality the discharge is decreasing. Then, again, a chinook causes a sudden rise in temperature and the discharge is often increased, while at the same time the gauge height gradually lowers, evidently because the warmer weather and water have melted out a lot of the ice from the channel and given it a greater carrying capacity.

In order to make reliable estimates of the daily discharge, gaugings must be made at short intervals and the weather conditions and temperatures in the whole of the drainage area above the stations must be very carefully studied.

The weather conditions and temperatures at the gauging station are not always typical for the whole drainage basin above, and care must, therefore, be taken to have the meteorological observations made at some other place, or, if necessary, at two or more places. Of course, care must be taken to study all the possible conditions which may affect the estimates.

LETTER TO THE EDITOR.

Stresses in Lattice Bars of Channel Columns.

Sir,—The writer was much interested in the discussions by Mr. Goodrich, Professor O. H. Basquin and others. I take it from their articles that they are just as anxious as myself to have this problem settled, and therefore wish to bring out clearly the principle upon which my discussion is based. I have assumed the following:—

1st. As the length l approaches zero, the stress in the column due to bending also approaches zero.

2nd. As the length l becomes greater, then the stress in the column due to bending also increases.

3rd. That it is only the stress due to the bending of the column that causes any material stress in the lattice bars.

4th. That all columns centrally loaded have a tendency to bend into the shape of a sinusoid whose equation is $y = \Delta \sin n \pi \frac{x}{l}$ as shown in Figs. 1, 7 and 8. (See Merriman's "Mechanics of Material," 1894 edition, page 115.) [Editor's Note—Figs. 1 to 6, inclusive, and Equations 1 to 18, inclusive, appeared in the February 24th, 1916, issue of *The Canadian Engineer.*]

5th. If we know the flange stress in the centre of the column due to bending, and plot a sinusoid whose centre ordinate is equal to that stress, then any ordinate taken at any point x (see Fig. 9) will be equal to the stress in the channel at that point due to bending.

6th. If the column (hinged top and bottom) is divided into, say, twelve equal parts, then the ordinates will have the relative proportions to the centre ordinate, as shown in Fig. 9.

If the six assumptions given above are correct, the problem resolves itself into a very simple one. If the column is hinged each end (or round ends) then the equation is

$$y = \Delta \sin \pi \frac{x}{l} \text{ (See Figs. 1 and 9.) [Equation 19.]}$$

If the column is hinged at bottom and fixed the other,

$$y = \Delta' \sin \frac{3}{2} \pi \frac{x}{l} \text{ (See Fig. 7.) [Equation 20.]}$$

If the column is fixed both ends,

$$y = \Delta'' \sin 2\pi \frac{x - \frac{l}{4}}{l} \text{ (See Fig. 8.) [Equation 21.]}$$

The case under discussion is the one shown in Figs. 1 and 9, Equation 19.

Now, referring to the 5th assumption, all that it is necessary to do is to substitute for Δ in Equation 19 the stress in each channel due to bending of column at the centre of the column. One will then get ordinates as shown in Fig. 9.

The ordinate y at any distance x will then be the stress in the channel at that point due to the bending of the column. If a stress curve is plotted as shown in Fig. 9, then it is possible to get the stress in any lattice bar.

The problem now resolves itself into getting the most correct column formula for the stresses in columns.

If the same notation is followed as called for in my February 24th article, S_c = 16,000 lbs. per sq. in. (safe pressure per sq. in.); S = 50,000 lbs. per sq. in. (ultimate pressure per sq. in.); E = 29,000,000 (modulus of elasticity). (S and E are values given in Cambria and Carnegie handbooks.)

$$\text{Then, } S' = \frac{S_c}{1 + \frac{S}{m\pi^2 E} \cdot \frac{l^2}{r^2}}. \text{ [Equation 22.]}$$

Substitute values given above and $m = 1$ for round ends, then

$$S' = \frac{16{,}000}{1 + \frac{1}{5{,}800} \cdot \frac{l^2}{r^2}}. \text{ [Equation 23.]}$$

(NOTE—Equation 23 is very nearly the same as that which was suggested by Mr. Goodrich, and is what Professor Merriman gives.)

Referring to Equation 19, we get

$$y = \Delta \sin \pi \frac{x}{l}.$$

Now, if in this equation we substitute for Δ the value

$$(S_c - S_1) \frac{A}{2},$$

the equation becomes

$$y = (S_c - S_1) \frac{A}{2} \cdot \sin \pi \frac{x}{l} \text{ (See Fig. D.) [Equation 24.]}$$

It will be noticed that Equation 24 is the same as Equation 16, except that I have substituted $\frac{A}{2}$ for A. This error was pointed out by Professor Basquin, and also by Mr. Goodrich.

Equation 24 gives the equation of the curve shown in Fig. 9 and similar to that shown in Fig. 1.

If ϕ is the angle the lattice bars make, then the stress in the end lattice bar $b\,c$ =

$$(S_c - S_1) \frac{A}{2} \sin \pi \frac{x}{l} \sec \phi. \text{ [Equation 25.]}$$

There are two lattice bars, so the stress in each end lattice bar becomes

$$b\,c = \frac{(S_c - S_1) \frac{A}{2} \sin \pi \frac{x}{l} \sec \phi.}{2} \text{ [Equation 26.]}$$

But ϕ is usually 60°, and sec of 60° is 2,

$$\text{Therefore, } b\,c = (S_c - S_1) \frac{A}{2} \sin \pi \frac{x}{l}. \text{ [Equation 27.]}$$

Equation 27 is the same as given in my February 24th article under Equation 16, except that $\frac{A}{2}$ has been substituted for A.

(NOTE—y given in Equation 24 is the stress in the channel at any point x, but as ϕ is usually 60°, and secant of 60° is 2, also as there are two lattice bars, therefore when $\phi = 60°$, then the stress in each end lattice bar $b\,c = y$, when x is the distance to the first rivet from the end.)

Referring to Fig. 9 and the 5th assumption.

The stress in the centre of the column in each channel will be $(S_c - S') \frac{A}{2}$.

If this quantity is taken as the centre ordinate and a sinusoid plotted, then the stress in the channel at any point x will be equal to the ordinate y and will be the stress shown as p in the end panel.

This assumption is based on the same theory as an ordinary lattice truss having an evenly distributed load in which, if the centre ordinate is made equal to the stress in either flange and a parabola is plotted, then the stress at any other point x is obtained by simply scaling off the ordinate.

It seems reasonable that if the stress in an ordinary lattice truss is proportional to the deflection curve, then the same law should apply to columns.

Referring now to Fig. 7.

The same method may be adopted by using Equation 20, and for Δ' substitute $(S_o - S')\,\frac{A}{2}$, finding S' by Equation 39.

Referring to Fig. 8.

Equation 21 must be used and for Δ' substitute $(S_o - S')\,\frac{A}{2}$, finding S' by Equation 43.

Transverse shear may be found as follows:—

Referring to Fig. 1. M = bending moment at centre of column. Then,

$$M = P\Delta. \quad \text{[Equation 29.]}$$

Total stress in one channel due to M is

$$(S_o - S_1)\,\frac{A}{2},$$

Therefore, $\frac{P\Delta}{D'} = (S_o - S_1)\,\frac{A}{2}$,

and $P\Delta = D'\,(S_o - S_1)\,\frac{A}{2}$. [Equation 30.]

Also, $m = Py$. [Equation 31.]

Substitute for y the value given in Equation 19, then

$$m = P\Delta \sin \pi\frac{x}{l}. \quad \text{[Equation 32.]}$$

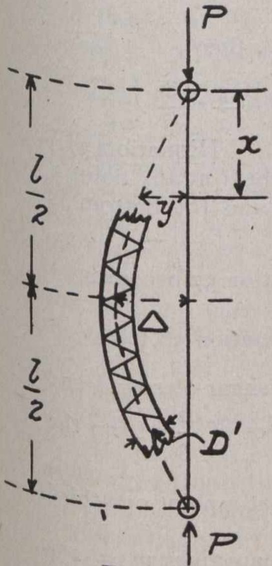


Fig. 1.
Round ends.
$n = 1$.

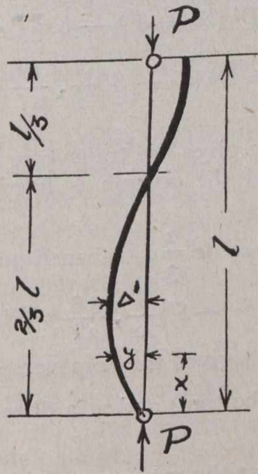


Fig. 7.
Top fixed,
bottom round.
$n = 3/2$.

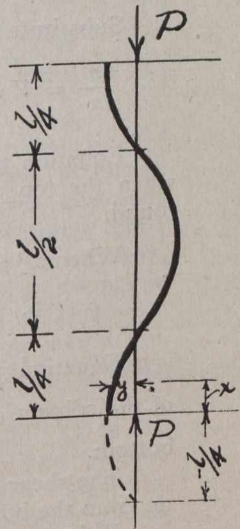


Fig. 8.
Both ends
fixed.
$n = 2$.

We know that $\frac{dm}{dx} = V$, where V is the shear. Differentiating Equation 32 we get

$$dm = P\Delta \cos \pi\frac{x}{l}\;\frac{\pi\,dx}{l}, \text{ or}$$

$$\frac{dm}{dx} = P\Delta\,\frac{\pi}{l} \cos \pi\frac{x}{l} = V. \quad \text{[Equation 33.]}$$

Equation 33 gives the transverse shear at any point x, and if $x = 0$, we get

$$V = P\Delta\,\frac{\pi}{l}. \quad \text{[Equation 34.]}$$

If, in Equation 34, for $P\Delta$ is substituted value found in Equation 30, then Equation 34 becomes

$$V = D'\,(S_o - S_1)\,\frac{A}{2}\,\frac{\pi}{l}. \quad \text{[Equation 35.]}$$

Equation 35 gives transverse shear; round ends.

Referring to Fig. 10, Cambria gives the following values with reference to axis 2-2:—

7″ channel	$D' = 6\frac{3}{4}''$	$r = 2.8$	$A = 5.7$
10″	"	$D' = 9\frac{1}{4}''$	$r = 3.83$	$A = 8.92$
12″	"	$D' = 11\frac{1}{4}''$	$r = 4.64$	$A = 12.06$
15″	"	$D' = 13\frac{1}{4}''$	$r = 5.61$	$A = 19.8$

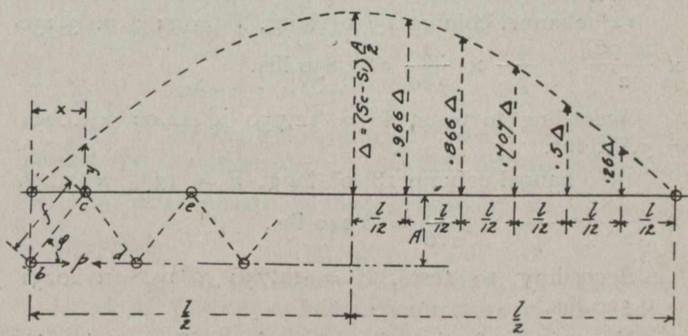


Fig. 9.

When $l = 9'\,0''$, then for 7″ channel columns,

$$\frac{l}{r} = 38.6, \quad \frac{l^2}{r^2} = 1{,}490.$$

When $l = 12'\,0''$, then for 10″ channel columns,

$$\frac{l}{r} = 37.6, \quad \frac{l^2}{r^2} = 1{,}414.$$

When $l = 15'\,0''$, then for 12″ channel columns,

$$\frac{l}{r} = 38.8, \quad \frac{l^2}{r^2} = 1{,}465.$$

When $l = 18'\,0''$, then for 15″ channel columns,

$$\frac{l}{r} = 38.6, \quad \frac{l^2}{r^2} = 1{,}490.$$

The above values for $\frac{l}{r}$ are very nearly what is given in Bulletin No. 44 for column No. 1, which was $\frac{l}{r} = 37.8$.

Substituting above values in Equation 23, the following is found for S':—

7″ channel column 9′ 0″ long, $S' = 12{,}730$;
therefore $(S_o - S_1) = 3{,}270$.
10″ channel column 12′ 0″ long, $S' = 12{,}850$;
therefore $(S_o - S') = 3{,}150$.
12″ channel column 15′ 0″ long, $S' = 12{,}750$;
therefore $(S_o - S') = 3{,}250$.
15″ channel column 18′ 0″ long, $S' = 12{,}730$;
therefore $(S_o - S_1) = 3{,}270$.

Substituting above values for D' and $(S_o - S')$ in Equation 35, we get for the transverse shear V, the following:—

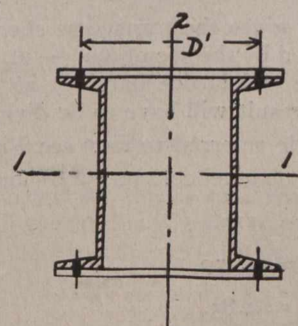


Fig. 10.

7″ channel column 9′ 0″ long, $V = 6\frac{3}{4} \times 3{,}270 \times \frac{5.7}{2} \times \frac{22}{7} \times \frac{1}{108} = 1{,}830$ lbs.

According to tests, $V = 12{,}730 \times 5.7 \times .0251 = 1{,}820$ lbs.

10″ channel column 12′ 0″ long, $V = 9\frac{1}{4} \times 3{,}150 \times \frac{8.92}{2} \times \frac{22}{7} \times \frac{1}{144} = 2{,}836$ lbs.

According to tests, $V = 12{,}850 \times 8.92 \times .0251 = 2{,}877$ lbs.

12″ channel column 15′ 0″ long, $V = 11\frac{1}{4} \times 3{,}250 \times \frac{12.06}{2} \times \frac{22}{7} \times \frac{1}{180} = 3{,}850$ lbs.

According to tests, $V = 12{,}750 \times 12.06 \times .0251 = 3{,}859$ lbs.

15″ channel column 18′ 0″ long, $V = 13\frac{1}{4} \times 3{,}270 \times \frac{19.8}{2} \times \frac{22}{7} \times \frac{1}{216} = 6{,}240$ lbs.

According to tests, $V = 12{,}730 \times 19.8 \times .0251 = 6{,}320$ lbs.

Now, for a further demonstration, if we take the column that was actually tested,

$$\frac{l}{r} = 37.8,$$
$$l = 21' \; 0'',$$
$$D' = 16,$$
$$A = 18.76,$$
$$\frac{l^2}{r^2} = 1{,}429,$$
$$S' = 12{,}840,$$
$$(S_c - S') = 3{,}160.$$

Then $V = 16 \times 3{,}160 \times \frac{18.76}{2} \times \frac{22}{7} \times \frac{1}{252} = 5{,}915$ lbs.

According to test, $V = 12{,}840 \times 18.76 \times .0251 = 6{,}046$ lbs.

Referring to the above, it seems to the writer that the formula given in Equation 35 for the transverse shear certainly agrees to a surprising degree with the figures for the transverse shear given in Bulletin No. 44, which gives ratio of transverse shear to compression load = .0251; and, as pointed out before, this was for a column having $\frac{l}{r} = 37.8$ and the slope of the lattice bars with axis of column = 63° — 30′.

It hardly seems reasonable that the above is only a coincident.

Now, to prove that Equation 27 and Equation 35 will give the same results:—

Equation 27 gives the stress in the end lattice bars when $\phi = 60°$.

Equation 35 gives the transverse shear and therefore must be multiplied by the sec of $(90° - \phi)$. This will give the stress in the end lattice bar, but as there are two lattice bars this result will have to be divided by 2.

For the angle referred to as ϕ see Fig. 9.

Take the 7″ channel column, 9′ 0″ long.

$$\sin \pi \frac{x}{l} = .113.$$
$$(S_c - S_1) = 3{,}270.$$
$$\frac{A}{2} = 2.85.$$

Then, using Equation 27.

$bc = 3{,}270 \times 2.85 \times .113 = 1{,}053$ (stress in end lattice bar bc).

Using Equation 35.

$$V = 1{,}830.$$

Secant 30° = 1.155.

Then, stress in each end lattice bar $bc = \frac{1{,}830 \times 1.155}{2} = 1{,}056$.

If 15″ channel column 18′ 0″ is taken,

$$(S_c - S_1) = 3{,}270.$$
$$\frac{A}{2} = 9.9.$$
$$\sin \pi \frac{x}{l} = .111.$$

Using Equation 27,

$$bc = 3{,}270 \times 9.9 \times .111 = 3{,}590.$$

Using Equation 35,

$$V = 6{,}240.$$

Then, stress in each end lattice bar $bc = \frac{6{,}240 \times 1.155}{2} = 3{,}600$.

Referring to Equation 20,

$$y = \Delta' \sin \frac{3}{2} \pi \frac{x}{l}. \quad \text{[Equation 20.]}$$

(Column fixed at top and round at bottom. See Fig. 7.)

$$m = Py, \text{ substitute for } y.$$

Therefore $m = P\Delta' \sin \frac{3}{2}\pi\frac{x}{l}$

$$\frac{dm}{dx} = P\Delta' \frac{3\pi}{2l} \cos \frac{3}{2}\pi\frac{x}{l}. \quad \text{[Equation 36.]}$$

Substitute $D'(S_c - S')\frac{A}{2}$ for $P\Delta'$, then

$$\frac{dm}{dx} = V = D'(S_c - S')\frac{A}{2}, \frac{3\pi}{2l} \cos \frac{3}{2}\pi\frac{x}{l}. \quad \text{[Equation 37.]}$$

Equation 37 gives the transverse shear at any point x when the top of the column is fixed and the bottom is round.

When $x = 0$, or $\frac{2}{3}l$ then Equation 37 becomes

$$V = D'(S_c - S')\frac{A}{2}, \frac{3\pi}{2l} \quad \text{[Equation 38.]}$$

Equation 38 gives the transverse shear at round end of the column (bottom) and at a distance of $\frac{2}{3}l$ from the bottom.

The shear at the top is zero, also at a point one-third up from the bottom.

Referring to Equation 22, Merriman gives $m = 2\frac{1}{4}$ when columns are fixed one end and round the other.

Substitute this value for m and values for S and E, then

$$S' = \frac{16{,}000}{1 + \frac{1}{13{,}000}\frac{l^2}{r^2}}. \quad \text{[Equation 39.]}$$

Referring to Equation 21,

$$y = \Delta'' \sin 2\pi \frac{x - \frac{l}{4}}{l}. \quad \text{[Equation 21.]}$$

Column fixed top and bottom. (See Fig. 8.)

$$m = P\Delta'' \sin 2\pi \left(\frac{x - \frac{l}{4}}{l}\right)$$

$$\frac{dm}{dx} = P\Delta'' \frac{2}{l} \cos \pi \left(\frac{4x - l}{2l}\right) \quad \text{[Equation 40.]}$$

Substitute $D'(S_c - S')\frac{A}{2}$ for $P\Delta''$, then

$$\frac{dm}{dx} = V = D'(S_c - S')\frac{A\pi}{l} \cos \pi \left(\frac{4x - l}{2l}\right) \quad \text{[Equation 41.]}$$

Equation 41 gives the transverse shear at any point x when both ends of the column are fixed.

If $x = 0$, then $\left(\cos \pi \dfrac{4x - l}{2l}\right) = \cos 90^\circ = 0$ and the end shear $V = 0$.

If $x = \dfrac{l}{4}$, then $\left(\cos \pi \dfrac{4x - l}{2l}\right) = \cos 0^\circ = 1$, and the shear becomes

$$V = D' (S_c - S') \frac{A\pi}{l}. \quad \text{[Equation 42.]}$$

Equation 42 gives the transverse shear at a distance $\dfrac{l}{4}$ from each end of the column, which is a maximum.

Referring to Equation 22,

$$m = 4.$$

Substitute this value for m, and values for S and E. Then,

$$S' = \frac{16{,}000}{1 + \dfrac{1}{23{,}200} \dfrac{l^2}{r^2}}. \quad \text{[Equation 43.]}$$

This value of S' to be substituted in Equation 42.

The value of the shears V, obtained in Equations 35, 38 and 42 should be multiplied by the secant of the angle $(90^\circ - \phi)$ and then divided by 2 to give the stress in the end lattice bars bc.

Reply to Mr. Goodrich.—Referring to Mr. C. M. Goodrich's discussion, would say that he was right in his criticism that A should be $\dfrac{A}{2}$, and I have corrected this above.

He mentions that the lattice bars are too wide at $2\frac{1}{4}''$ and suggests $1\frac{3}{4}''$ for the small columns. This, of course, is a matter of shop practice and depends a great deal upon the size of the rivet that is being used. He states that r is taken for the wrong axis. Upon looking up Cambria, and checking over the r used, I find that in a few places the value for r may be wrong, such as for $7''$ channel I took 2.34, and upon carrying the calculations out to the fourth place I find that this should be 2.38, but I am sure this would not affect the results materially.

He says that Mr. Pritchard suggests using 3% of the axial stress, and further on states that the new Quebec bridge lattice takes a shear of 2% of the axial stress, but he doesn't in any place state upon what authority 3% or 2% was taken, nor does he say whether he agrees with this assumption or not, or whether he thinks this value should be taken for all lengths of columns or how the columns should have their ends fixed when the above values are used.

If Mr. Goodrich believes there is a transverse shear equal to 2% of the axial load, then he must also believe there is bending in the column, for we know that

$$\frac{dm}{dx} = V,$$

where V is the vertical shear.

Or, in other words, when there is any possibility of the member acting in any way as a beam, it is not possible to have a shear without a moment except as dx approaches zero.

Therefore, if V is 2% of the load on the column (centrally loaded), it must necessarily follow that it must be a function of the bending of the column, even though the column be less than a ratio of $200 \dfrac{l}{r}$.

Upon referring to the report of the Royal Commission, Quebec bridge inquiry, I find that Mr. Schneider has assumed that the column will bend into shape of a parabola due to eccentricities of load caused by fabrication, and he has given a formula for the transverse shear as follows:—

$$S_{max} = 8C \frac{ar}{d}.$$

Where $C = 70$, d = out to out of flanges; this then resolves itself into

$$S_{max} = \frac{280\,ar}{n},$$

$$\text{where } n = \frac{d}{2}.$$

This is the same equation as given in my first discussion for Equation (d). I also find that the particular member under discussion had a ratio of $\dfrac{l}{r} = 35$, approximately.

Reply to Mr. Harkness.—Referring to the discussion by Mr. A. H. Harkness, he has pointed out the same error as previously mentioned.

If you take my original Equation 11, which is

$$M = P\Delta \sin \pi \frac{x}{l},$$

and differentiate this, we get

$$dm = P\Delta \cos \pi \frac{x}{l} \cdot \frac{\pi\,dx}{l},$$

$$\text{or } \frac{dm}{dx} = P\Delta \frac{\pi}{l} \cdot \cos \pi \frac{x}{l},$$

which is the transverse shear on the column, and this equation is the same as Equation (2) given by Mr. Harkness. He has, then, substituted for $P\Delta$, the quantity

$$f \frac{Ar^2}{n}.$$

I think his analysis is a neat treatment of the subject, and, as he says, a quicker method of arriving at the result.

Reply to Mr. Molitor.—Mr. Molitor is of the opinion that, due to imperfections of fabrication, it would be impossible to get an exact formula for the stresses in lattice bars. I thoroughly agree with him, and pointed that fact out in my article, but I do think it possible to arrive at a

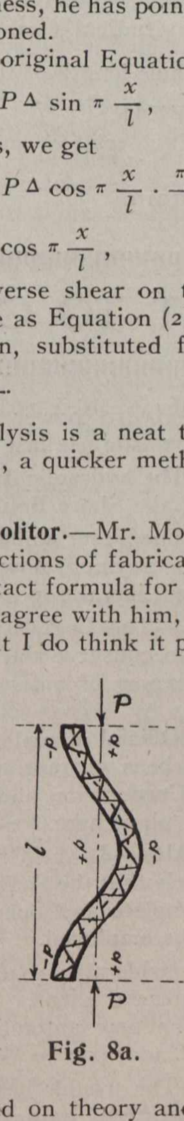


Fig. 8a.

formula that is based on theory and that will give reasonably safe results, and this was my object.

Mr. Molitor derives the old formula,

$$R = \frac{280\,ar}{n},$$

but afterwards discards it, and states in his last paragraph the following:—

"These flanges being subject to compression from end to end, the function of the lacing and batten plates will be to transfer longitudinal shear *from one flange to the other* whenever and wherever the compressive stress is unequally distributed. The maximum value of this shear may, under certain conditions, approach the total load P on the column and hence the total area of lacing and batten plates should be sufficient to carry the shear. This will serve as a good criterion in designing lattice columns and represents the writer's practice, though the whole subject is largely a matter of standards."

I would draw attention to Fig. 8a, which shows a column fixed top and bottom. It will be noticed that there is a compressive stress at the top and bottom, designated by $+ p$, and this, then, appears at the centre on the opposite side. Now, this stress $+ p$ is due to the bending of the column, and might, as Mr. Molitor says, approach the size of P if the deflection were great enough.

The stresses shown in Fig. 8a should fulfil Mr. Molitor's conditions excepting that one also gets a transverse shear as given by Equation 42. I do not quite see how Mr. Molitor could get a longitudinal shear without a transverse shear, and if the conditions that I suggest are right, then the longitudinal shear must be taken up by $\frac{l}{4}$, and not by the whole length of the column.

W. W. PEARSE,
City Architect and Supt. of Bldg.,
Toronto, Ont.

Toronto, April 27th, 1916.

COAST TO COAST

Petrolea, Ont.—Hydro power was turned on for street lighting and domestic use last week.

Toronto, Ont.—The hydraulic dredge "Cyclone" has started work on the Lake Shore Boulevard scheme.

Toronto, Ont.—The Trent Valley power bill, validating the purchase of the Seymour power interests, has been passed against strong Liberal opposition.

Edmonton, Alta.—The Legislature has made an important amendment to conserve the natural resources of the province. The export of natural gas has been prohibited.

Ojibway, Ont.—Official denial is given to reports which recently have been current to the effect that the United States Steel Corporation had abandoned its plans to erect a large new plant here.

Medicine Hat, Alta.—If the C.N.R. build their line here this year it is likely that the Saskatchewan Bridge and Iron Works will complete and operate their new plant. About 200 men will be employed.

Banff, Alta.—Considerable roadwork has been done in the Rockies by interned aliens. They have been constructing parts of the new automobile road which will ultimately run from Vancouver to Winnipeg.

Calgary, Alta.—The trouble between the C.P.R. and the farmers in the western irrigation block has been settled. An entirely new contract will be entered into by the settlers, of whom there are about 1,000 in the western block.

Edmonton, Alta.—A new hotel enterprise in the form of a tented city is to commence operations in the near future in Jasper Park. The government has built a network of good roads to points of interest in the park during the last year.

Port Moody, B.C.—The main building of the steel works is now completed and 26 car-loads of machinery is now on the ground while heaps of miscellaneous scrap iron and steel litters the ground in the immediate vicinity of the plant. About 50 men are employed.

Toronto, Ont.—The need of diverting Bloor Street to provide for the construction of a viaduct over the Humber River at some future date has been brought to the attention of the Board of Control by R. Home Smith. The matter has been referred to Works Commissioner Harris.

Montreal, Que.—The reason for the increase in the capitalization of the Nova Scotia Steel and Coal Company, which was authorized at the annual meeting a few weeks ago, is now becoming apparent. It is now believed that the company will soon embark upon the steel shipbuilding industry.

Montreal, P.Q.—The Canadian Research Bureau has been established by the Canadian Pacific Railway. Its purpose is to investigate and study the natural resources of the country in a scientific manner. Research laboratories will be established at different points in the Dominion.

Victoria, B.C.—According to Jos. J. Warren, president of the Kettle Valley Railroad, the line from Nelson to the coast will not be put in operation until July 1st. The delay has been caused, it is said, by the heavy snow in the interior. Only 1½ miles of line remain to be completed.

Peterborough, Ont.—The township of North Monaghan is doing roadwork on broader and more extensive lines this spring than it has attempted heretofore. The work for the most part consists of rebuilding and repairing gravel roads. A new road drag has been purchased by the authorities.

Toronto, Ont.—Dr. T. Kennard Thomson has outlined a scheme for a new power development at Niagara Falls. Dr. Thomson's scheme is to build a dam several miles below the Falls and thus preserve the natural beauties of the district, the water being used after it passes over the Falls.

Victoria, B.C.—The Provincial Government is considering a project for the development of the water power possibilities of the province on a large scale, according to an announcement by Hon. Lorne Campbell, Minister of Finance, who was the guest of honor at the annual meeting of the Victoria Board of Trade.

Welland, Ont.—At a joint meeting of the Council and the Hydro-Electric Power Commission a statement was made that the consumption of hydro power had reached 73,000 horse-power. This, it is said, is the greatest amount owned by any municipality in Ontario. The plant has showed a surplus ever since the first year of operation.

The Pas, Man.—Steel laying on the Hudson Bay Railway will be resumed on May 1st from mileage 242. The steel cantilever bridge at this mileage has been completed and trains will be operated over it shortly. There is a demand for railway laborers and 1,000 men are required to carry on the season's construction programme.

Winnipeg, Man.—The waters of the Red River have been in flood and damage amounting to thousands of dollars has been done. Sewers have been undermined and pavements have collapsed. Many buildings have been damaged by their cellars being flooded and foundations weakened. The railways operating out of here have had great difficulty in keeping their lines open.

VANCOUVER BRANCH, CANADIAN SOCIETY OF CIVIL ENGINEERS.

E. G. Matheson, M.Can.Soc.C.E., gave his postponed lecture on "Pneumatic Foundations" on April 13th last.

Editorial

ACCELERATED PAVEMENT TESTS.

Accelerated pavement tests are receiving the attention of a great many highway engineers and chemists, and it is quite likely that in a few years some method will be invented which will enable the engineer to ascertain to a greater degree of certainty, before be builds a pavement, just how long that pavement will last.

In a recent lecture at Columbia University, C. N. Forrest, chief chemist of the Barber Asphalt Paving Co., said that a series of tests which could be applied to the pavement in place would be more satisfying to the engineer than the type of tests now employed.

Admittedly this is true; but the suggestion seemed so far in advance of any practical methods that are available at the present time, that *The Canadian Engineer* inquired of Mr. Forrest whether he had any idea as to how such a series of tests could be made.

Mr. Forrest replies that he cannot figure out any test such as he suggests would be desirable if someone could kindly invent it. An interesting point in Mr. Forrest's reply, however, is that the design of physical tests, for laboratory use, upon the equivalent of the pavement in place, *i.e.*, compressed test specimens of the composition or mixture as a whole, is now fairly well worked out for sand and fine stone aggregates.

These tests, applied at extremes of temperature, indicate the relative value of different proportions of raw materials, so that the best combination thereof can be adopted as a standard for the entire work in which they are to be used. They also indicate the relative value of various proportions of any asphalt when considered in connection with the mineral aggregate that is available.

It is possible to determine the degree of plasticity of the mixture, *i.e.*, its capacity to resist pushing and its resistance to wear by attrition. Having selected the materials for the work, and the best proportions, the chief consideration in this test scheme, in getting the mixture in place in the street, is proper compression. A measure of compression is the specific gravity of a sample taken from the pavement after compression.

Chemical analysis of the mixture, penetration of the asphalt cement, etc., will serve for plant control, says Mr. Forrest. If such a plan of operation is sufficient for the purpose, then any failure in the street later on would probably be due to workmanship or to engineering details such as drainage or strength of concrete base.

SEWAGE DISPOSAL PLANTS NEEDED.

The city of Lethbridge, which has been suffering from an epidemic of typhoid fever this spring, has presented a memorial to the premier of Alberta asking that the government compel towns to construct sewage disposal systems. According to Dr. Jamieson, the provincial bacteriologist, who has been investigating conditions at Lethbridge, the cause of the epidemic there is due to a polluted water supply. The city of Lethbridge procures its water from the Old Man River, and it is chlorinated. It would seem, however, that the degree of chlorination has not been sufficient to prevent the present epidemic. Doubtless under normal conditions the amount of chlorine used would have been ample, but under conditions such as are alleged to have existed, it was insufficient.

Dr. Jamieson carefully examined all probable sources of infection. The milk supply naturally was among the first of the city's organizations to come under suspicion; but here no fault could be found. All the dairies serving the city were found to be free from bacteria. The water supply was the sole source of trouble. Water samples which had been sent to Edmonton several weeks before the epidemic broke out had been pronounced potable. Evidently, then, the water had been polluted in the interim. Old Man River and its tributary watershed were investigated. It was found that the town of Macleod had had an epidemic of typhoid during the months of October and November last year, and that they were dumping raw sewage into the river. Spring freshets washed downstream all the impurities that had been deposited during the winter.

It is only fair that all towns discharging sewage into streams that are used as sources of water supply should treat their sewage before doing so. If a town will not take precaution to safeguard the health of its neighbors by looking after its own disposal of sewage, then such measures as are suggested in the memorial prepared by the city of Lethbridge, are most certainly needed.

LETTER TO THE EDITOR.

Stresses in Concrete Arch Dams.

Sir,—Mr. W. Gore's remarks in *The Canadian Engineer* of March 30th, 1916, regarding the desirability of relying upon "initial stress" to assist the stability of a concrete arch dam have been carefully read. The writer takes pleasure in attempting to answer the questions, especially because it is evident that Mr. Gore has given the subject considerable study.

Suppose the "initial stress" does not assist the stability of such an arch dam, and this it does not in arch dams carelessly constructed, then more load would be thrown on the cantilever, some more load taken up by shear action in the lower portion of the dam along the foundation, and some additional load thrown on the arch towards the crest. On a high, slender section this condition may cause higher cantilever stresses than desired; that is, high compression at the toe.

Fortunately, however, full load is not thrown on such structures in an instant, but weeks or perhaps months are generally required to fill up the reservoir. During this interval the modulus of elasticity of the concrete has had time to adjust itself according to the different amount of stress thrown on different portions. Due to the action of the time factor (for concrete only), parts highly stressed such as the toe, deforms much more than in proportion to the load carried, and, of course, as the concrete yields, more load is transferred to some other place of lower stress, thereby relieving the most highly stressed part. That is one reason why Formula 8, page 321, is empirical. For the benefit of anyone interested in the action of the time factor upon stress and deformation, the writer wishes to refer to two papers read at the Twelfth Annual Convention of the American Concrete Institute in Chicago, February, 1916. One by Mr. Earl B. Smith, entitled "Concrete Flows Under Sustained Load," abstracted in The Engineering Record of March 4th, 1916. The other paper, entitled "Tests Showing Continued Deformation Under Constant Load," By Prof. A. H. Fuller and Prof. C. C. More, University of Washington, Seattle, Wash.

For carefully constructed dams the writer feels that the theory holds true. High, massive arches should be provided with contraction joints, say, 50 feet apart (on small dams these are not necessary as the section is more flexible), and the structure built up in alternate sections between contraction joints, the closing being effected during cold weather. The dam should not be built too fast, 20% plum stones should be used if practical. In addition, on important work, the contraction joints should be provided with grout pipes to facilitate the grouting of these joints under pressure during cold weather with reservoir empty after having been full at least once.

Such a dam structure is likely to follow the theory in its action; it is very likely to be absolutely watertight, and, of course, safe, if designed for reasonable stresses, say, less than 30 tons per square foot. According to the writer's view, careful construction is more important than the use of low stresses in the design. The concrete used should have a crushing strength of approximately 1,200 lbs. per square inch when 28 days old.

The writer wishes to point out the possibility of constructing an arch dam in such a manner that the stresses are actually better distributed than Formula 10, page 322, would indicate. This can be accomplished by using many plum stones in the concrete along the upstream face, and few or none along the downstream face, thereby making the shrinkage due to setting less along the upstream face, and also making the modulus of elasticity higher along this face, both of these conditions tending to effect a transfer of stresses from the downstream face towards the upstream face, and also tending to lessen the cantilever stresses at the downstream toe, making all stresses more uniformly distributed than the formulas would indicate they are.

LARS JORGENSEN.

San Francisco, Cal., April 11th, 1916.

OBITUARY.

COLIN McLEAN, one of the best-known contractors on the Atlantic seaboard, died at his home in Baltimore recently of pneumonia. He was born in Nova Scotia seventy-two years ago. Among his undertakings were the construction of the foundations for the Statue of Liberty and Brooklyn Bridge.

Dr. WILLIAM FREDERICK KING, chief astronomer of Canada and commissioner for the survey and marking of international boundaries, died on April 23, at the observatory residence, Ottawa, Ont. The late Dr. King was born at Stowmarket, Suffolk, England, 62 years ago, coming to Canada with his parents eight years later. He entered the service of the Dominion Government in 1872 as assistant astronomer on the North American Boundary Commission, and became inspector of surveys for the Dominion in 1881. He was made chief inspector of surveys in 1886 and chief astronomer of the Department of the Interior in 1890.

PERSONAL.

J. H. McMILLAN, of Cumberland, has been appointed inspector of mines with headquarters at Prince Rupert, B.C.

H. D. CAMERON has been appointed mechanical engineer of the Canadian Northern Railway, with office at Toronto, Ont.

JOHN AHEARN has been appointed superintendent of the Ottawa Street Railway. Mr. Ahearn has been with the company for 15 years.

A. J. RANDALL, formerly manager of the Saskatoon Iron Works, Saskatoon, Sask., has gone to Winnipeg, where he will take an officer's course.

G. V. HASTINGS has been appointed by the Winnipeg city council to succeed J. H. ASHDOWN as one of the commissioners on the Winnipeg and St. Boniface Harbor Board.

W. H. DINSMORE has been appointed traffic superintendent of the Vancouver city and suburban lines of the British Columbia Electric Railway, succeeding Mr. James Hilton, resigned.

FREDERICK KEFFER, mining engineer, Spokane, Wash., has left for Ashcroft, B.C., where he will take charge of constructing a concentrator for copper ores of the Highland Valley Company.

E. D. W. COURTICE, assistant superintendent of the John Street pumping station, city of Toronto, has resigned his position to enter the employ of the Hare Engineering Company, Limited, as assistant engineer.

R. S. LEA, M.Can.Soc.C.E., Montreal, has been in attendance at the session of the International Waterways Commission, Washington. Mr. Lea reported to the Commission on the level of the Lake of the Woods, Manitoba.

ARTHUR D. LITTLE will have charge of the new Canadian Research Bureau which is being established by the C.P.R. Mr. Little is a past president of the American Chemical Society and a member of the Institute of Chemical Engineers.

G. H. STEVENS has resigned his position as electrician-in-charge of the Fort Erie district for the Canadian Niagara Power Company, and commenced his duties as power apparatus specialist, Northern Electric Company, Montreal, on April 1st.

R. P. TRIMBLE, mining engineer, Portland, Ore., has returned from a trip to California and is leaving immediately for Telkwa, B.C., to commence development and take charge of operations at the Cassiar Crown Copper Company's property.

WILLIAM G. MURDOCH, city engineer of St. John, N.B., addressed the engineering students of the University of New Brunswick recently on the Suspension Bridge. His address was very instructive and much appreciated by the engineers.

Lieut. N. H. DANIEL, B.A.Sc., who left with the Divisional Cyclists Corps as a private and was later granted a commission in the Tenth Motor Machine Gun Battery, has been wounded. Lieut. Daniel is a graduate of S.P.S., Toronto, and was a member of the rugby team in his final year.

BLAIR RIPLEY, M.Can.Soc.C.E., who has had charge of C.P.R. grade separation in Toronto, has been appointed to command a new construction battalion with the rank of Lieutenant-Colonel. The battalion will be composed of men engaged in bridge building, railway construction, roadwork and general construction for overseas service.