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WATER SUPPLY FOR CITIES AND TOWNS

AN OUTLINE OF THE PROBLEMS CONNECTED WITH WATERWORKS CONSTRUCTION AND VALUABLE HINTS AS TO THEIR SOLUTION.

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AMONG the problems which attend the establishment of a water supply for a community the first, and often the most important, is its source, particularly in the case of towns and cities that are not so favored by nature as to be able to secure an ample and safe supply from adjacent lakes or rivers, but have to resort to wells, collecting grounds, etc.

The first step the engineer is called upon to decide is the selection of a suitable watershed and site for the proposed works, including the dam and reservoir. Since we depend solely upon rain for our supply of water, after these selections have been made it is necessary to find out exactly how much rain actually falls on the contemplated watershed drained by the stream which is to feed the impounding dam. To do this, rain gauges must be fixed on the watershed and from these an estimate made of the rainfall. After comparing them with any existing rain gauge records which have been kept over a number of years in close proximity to the proposed site, the engineer should be able to strike a fair average of the rainfall. The longer the period taken, the better and more satisfactory average results will be obtained. From these records an estimate of the quantity available for storage can be calculated allowing, of course, for losses on the average rainfall due to evaporation, percolation, and absorption, which allowance can only be estimated from previous experience on similar gathering grounds, and practical experience of the engineer. After computing the available supply for storage, the available yield in gallons per day is arrived at. An excellent check on the amount—a really better method to obtain the actual supply—lies in the use of stream gauges. They should be placed, wherever possible, in the feed channel supplying the impounding dam, and, when fitted with an automatic recorder, render the operation a simple and accurate test for the actual yield.

In most cases part of this supply will have to be liberated to supply rivers, etc., for compensation water (usually fixed at ⅓) and this must be deducted before deciding upon the average available daily supply.

The size of the impounding dam should be next settled. The late Thomas Hawkesley's well-known formula, which gives the number of days' storage required, taking for safety the mean annual rainfall for the three driest years, is recommended. Then, after levelling and contouring the site, the engineer can determine the height and dimensions of the dam to impound the required number of gallons. The actual construction of the dam will depend largely on local conditions, especially as to material of which it will be constructed, but the ground should be carefully examined, trial holes sunk, plans prepared showing all levels, etc., and geological formation before the actual locality and line of the proposed structure can be definitely settled.

In this paper it is not proposed to enter into the actual design of the dam and the many calculations required as the subject cannot be treated properly in the limited space, but on its stability the engineer is very often confronted with serious difficulties, and the most important points for his careful consideration will be the foundations, stability of walls, and a properly designed overflow to take off the flood water. A dam may fail by overturning, crushing, sliding, shearing, or by rupture due to tension, and to safely guard against all these causes of failure the section of the wall must be such that the lines of resultant pressure, both when the dam is full or empty, must fall within the middle third of any horizontal joint, in order that the maximum pressure on the foundation will never exceed certain fixed limits of safety. The friction between any horizontal layers, into which the dam may be divided, and also between the main walls and their foundation must be sufficient to prevent any sliding taking place. The ideal and safe cross-section of wall is the one which is constructed of sufficient dimensions to safely withstand all these pressures. After they have been properly calculated, the method adopted by the author in calculating the required sections for the design of the walls is by mathematics, and on completing same, he re-checks graphically, so that the line of pressure must fall within the middle third.

A considerable economy can be made if the site of the valley across which it is proposed to construct the dam is narrow, by making the dam curved and reducing the section, but the author does not recommend, from a stability standpoint, a greater radius than 300 feet being actually constructed. One of the main points, frequently overlooked, and upon which the stability of the dam and safety of the system so largely depend, is the provision of a sufficiently large overflow to amply take care of all likely flood water, as without such the works may be seriously damaged and unnecessary trouble and expense incurred which, at little first cost, could easily be provided for. From the author's own observations and experience he advises a 3 to 4-ft. length of weir to every 100 acres of watershed, fixing the maximum height, in all cases, to which the water is allowed to rise above the crest of the weir at 1 ft. 6 in. to 2 ft.

Purification of Water.—The methods adopted to purify water for domestic consumption are æration, subsidence, precipitation, straining, and filtration.

Aeration or oxidation is sometimes adopted to get rid of certain impurities in the water, such as dissolved sulphuretted hydrogen, exposure to the atmosphere in thin sheets or sprays, has the effect of also softening hard waters, by releasing the loosely combined carbonic acid and precipitating the carbonate of lime. This process is very rarely adopted, the chief objection being vegetable growth, but the action of oxygen in the atmosphere on water in rivers or channels, certainly gradually oxidizes the organic impurities, and in time renders them innocuous and the water more palatable.

Subsidence.—This procedure generally takes effect in storage reservoirs and depends solely on the specific gravity or fineness of the matter in suspension being deposited.

Precipitation.—Water, especially when obtained from bore holes (see later) is often hard and sometimes it is of such a nature that after standing it becomes slightly discolored, due generally to deposit of iron. Now that the question of an ample and suitable water supply plays such an important part in our present-day manufacturing industries, it is essential from this standpoint, as well as from the domestic one, that the supply should be softened and purified to be beneficial for all concerned. Without entering into any details of any individual water-softening process, the general outline of the cheapest and universal methods and materials adopted in most cases, may be briefly given. Caustic lime is one of the most economical precipitants and the *modus operandi* is to add a certain quantity of lime to a measured volume of water in a tank or vessel, to form lime water. The clear fluid is then drawn off by a float pipe into another tank and mixed with the water to be softened. The result is the formation of carbonate of lime, which is precipitated along with the carbonates already in solution. In some plants it is an advantage to discharge "purified" generated carbonic acid gas into the main delivery pipe from the softening tanks. If, at the expiration of the time allowed for settling, there are any particles of lime left in the water, whether visible or otherwise, they unite with the carbonic acid gas and form a soluble carbonate which is taken up by the water and does not make any appreciable effect on the hardness of the softened water.

Straining and Filtration.—Straining water through fine screens of brass or copper set in wooden frames has proved satisfactory, and in some cases the author recommends their use in reservoirs, as, when properly supervised and cleansed by a jet of water, they arrest and intercept large quantities of floating suspended matter. Gravity or sand filters have been adopted in the past, owing to the general opinion that the slow gravitation of water through layers of sand, polarite, spongy iron and other media removes the suspended matter and purifies the water better than any other system, and the author under this head intends to illustrate the many advantages obtained by the installation of suitable mechanical filters in preference to the gravity type, especially in cases of large cities. A mechanical plant does not require the extensive area of a gravity bed; consequently there is a great saving in purchasing suitable land. A million gallons per day can be treated on gravity filters on an area of approximately 2,000 sq. yd., whereas the same quantity could be mechanically treated on a space of 60 to 70 sq. yd. This is a considerable saving, especially in works of any magnitude, such as required for large cities. Again, whatever a filter bed extracts it retains and the better the filtering material the sooner it becomes clogged and requires cleansing. Therefore, ease and rapidity of cleansing should play an important point before the engineer decides on the class of filters he should adopt.

The gravity beds, when downward filtration is adopted, requires the constant attention and removal of the top layer of sand, containing the perceptible suspended matters. If the sand is costly and has to be washed, an expensive sand-washing machine is centrally installed which necessitates a cleansing gang continually employed to wash the beds in rotation. On large filtration areas this is a very costly maintenance item.

The cleansing of the mechanical filters is simply accomplished by upward or reversed flow of water, aided by the complete agitation of the filtering media by revolving arms, and the greater the amount of agitation within the bed of filtering material, the more rapid and effectual is the cleansing. The actual time taken in cleansing one of these filters varies from 3 to 5 minutes. For several obvious reasons the filtered water should only be used for cleansing purposes, as the impurites of unfiltered or raw water would collect at the bottom for the time being, and would have to be removed before considered fit for use.

A mechanical filtration plant capable of dealing with 5,000,000 gal. per day should take approximately 1½ hours to clean, using about one gallon to every 300 gallons of filtered water. Cleansing in mechanical filters for successful results should be accomplished every two days, which goes to prove that the filter is doing efficiently that which it is intended for. Another big advantage of mechanical filtration over sand filtration lies in the compactness of the plant, which facilitates better and cheaper maintenance, also a more complete and concentrated control over the feed pipes to the service reservoir.

It has been found and adopted in practice that the introduction of aluminum sulphate is very beneficial from two important aspects: (1) It acts as a coagulant causing minute particles to unite and become one large particle; (2) being converted into aluminium hydrate, a gelatinous insoluble mineral matter is formed on the filter bed, being impervious to micro-organisms, and having the power to extract coloring matter, adds materially to the filtering media, as the coloring is often due to dissolved vegetable matter and cannot be arrested in the finest filtering material unless in conjunction with precipitated aluminum-hydrate. This would not be feasible or workable in cleansing operations for gravitation or sand-bed filters.

There are several different mechanical filtration plants from Great Britain and the United States well represented on the Canadian markets, and the author advises any local authority who contemplates the installation of mechanical filters to place the scheme in the hands of an expert water engineer who can get out the necessary data and specifications for such an installation, and is in a better position to judge technically the merits and cost of such installations represented by the different companies who are invited to tender.

Service Reservoirs.—Service reservoirs are supplied direct from the impounding reservoir, or where filtration is necessary from the filter beds. Their use in water supply systems is to regulate the variation in daily consumption, and during accidents or break-downs to supply sufficient storage to meet the supply requirements. Their size, or storage capacity, depends on local conditions of the scheme, such as the distance from source of supply, single or duplicated feed pipes (especially important in case of a pumped supply). As a general rule, 2 to 3 days' storage capacity should meet all emergencies.

The site of reservoir should be at a sufficient elevation to give the required pressure on the mains and as close as convenient to the district to be supplied. The actual construction of it depends on local conditions, both from a material and design standpoint. The author favors concrete construction with puddle clay backing and entirely covered in with arch construction, well ventilated and having at least 2 to 3 feet of earth filling above the roof to keep the water as cool as possible. The tank should be subdivided into at least two portions for cleansing purposes. The reservoir should be provided with ample arrangements for inspection and cleansing and the inlet, outlet and overflow pipes placed in the best position the local demands and requirements call for. After passing through the reservoir, the water should flow through a meter house constructed at the head of the main distribution pipes to the city and correct records kept by means of a meter recording automatically the daily consumption and delivery of water into the city at all hours of the day.

Wells.—A difficulty often arises in finding near a large town a suitable gathering ground. In such cases the engineer directs his attention to a source of supply from wells, if possible. In selecting the site and position of bore-holes, the engineer, if not conversant in practical geology, should engage the services of an expert geologist before finally selecting and commencing on a scheme. The preliminaries mentioned previously as regards rainfall calculations should be carefully and accurately taken and records kept. The construction for the bore-holes present very few engineering difficulties and is accomplished by the several drilling and deep-well boring outfits. The author has had experience in both the "free-falling tool" and the "slack rope" method, and can recommend both as worth consideration, the local conditions governing the final selection. Perhaps details the writer has at hand of the actual construction of a well and bore-hole for a large city in Great Britain will be of interest, as it gives an excellent idea of the usual procedure. The well was first constructed 8 feet inside diameter, 24 feet deep with 14 inches of best hard blue brick lining filled in behind with cement grout. The floor consisted of 18 inches of cement concrete. A circular cast iron stand-pipe, 29-in. bore 9 feet long with flanged end to which was fixed during boring operations two similar lengths bolted on with special short valve casting to facilitate the testing of the yield of the bore-hole at certain stages. The bore-hole for approximately 110 feet was 28 in. in diam. and lined with 25-in. bore steel tubes, the annular space being filled in with concrete and so lined to prevent the contamination by impure surface water. The boring was continued from the lining tubes with a 14-in. circular hole to the rock strata, a distance of approximately 600 feet, when the water-bearing strata was penetrated sufficiently for the following tests of the probable yield of water to be made. The previously mentioned valve at the top of the stand-pipe was closed and the brick well emptied of all water, carefully measured, and the water admitted by opening the valve on the standpipe. The time taken in filling the well was accurately measured as the water rose to the surface of the ground. This level was then taken as the rest level and surface pumping was continued to lower it. From these results the actual yield of water flowing into the well per hour was ascertained by hydraulic formula. The head on the valve being known, the yield in gallons per 24 hours could readily be calculated. In this particular case the above was considered sufficient to warrant the installation of the pumping and machinery plant. The actual yield from the bore-hole was almost 750,000 gal. per day.

Distribution.—It is always the aim of the engineer to install a gravitation system, not only from an initial outlay standpoint but also owing to maintenance cost. Providing the selected site for supply reservoir is situated at such an elevation as to give the minimum pressure for supply purposes, both for domestic and trade use, this can be adopted; but should it not be available, pumping machinery will have to be installed. It requires careful thought in designing a complete and suitable pumping plant of sufficient capacity to supply the maximum demand at all times and at the most economical cost, having in mind local conditions and requirements. The author has visited waterworks plant where considerable saving has been accomplished by means of syphons. A great deal of costly excavation can be saved by adopting syphons providing for practical working purposes the difference of elevation between the summit and level of water from which the supply is to be drawn does not exceed 25 feet (34 feet theoretically).

The syphon should be provided with an air pump attachment on the longer leg, and in all cases for proper and satisfactory results, sluice valves and cocks must be placed in proper positions.

Water is conveyed generally under pressure from the source of supply to the consumer and pipes are mostly used. They have been constructed in cast iron, wrought iron, steel, wood, and reinforced concrete. Whichever material is adopted the pipes should be properly calculated for diameter and thickness to give the delivery and strength required. The thickness of metal in pipes requires careful consideration and good judgment on the part of the engineer, as after calculations have been made, sufficient allowance must be made for imperfect workmanship, shocks in handling and laying, weight of earth and traffic, also the great strain to which pipes are subjected on the sudden closing and opening of valves. It is the author's practice in calculating the required thickness from formula to withstand the water pressure, to allow a high factor of safety to take care of the above-mentioned additional strains, and in no case using less than ⅝-in. in thickness whatever the calculations work out at.

A margin on the diameter should be allowed for possible corrosion. In specifying cast iron pipes, they should be cast vertically and dipped when hot into a hot bath of a solution consisting of asphalt, resin, pitch and linseed oil. A percentage of test bars 3 in. x 2 in. x 3 ft. 6 in. should be cast at the same time and tested for tensile strength and deflection. The following tests by Sir Frederick Bramwell, strikingly illustrate the increase in strength and density obtained by re-melting the metal before finally running:

Samples.	Tensile Strength per sq. in.
1st sample	7.5 tons
2nd sample, after 2 hours longer fusion	8.3 tons
3rd sample, after 1¾ hours longer fusion	10.8 tons
4th sample, re-melted with fresh pigs	11.0 tons
5th sample, after 4 hours longer fusion	18.5 tons
Maximum of 5th sample	19.6 tons

The strength at the spigot end is increased by casting an additional 6 in. or more beyond the finished length of the pipe, this having the effect of compressing the metal and permitting ash and air bubbles to rise into same, which extra length is finally cut off in the lathe and the pipe finished off to required dimensions. All pipes should be tested by oil or water (preferably the former) in a testing machine up to at least twice the pressure they will afterwards be subjected to before leaving the foundry.

Wrought iron and steel pipes have of late found favor amongst engineers, especially for large mains, because of their greater tensile strength and lightness over cast iron giving them many advantages in cases where weight and strength are the main objects, but owing to the thickness and rapid corrosion of wrought iron and steel pipes the author thinks cast iron will be hard to displace for general use, as they are cheaper and more easily made Their greater thickness allows for corrosion, which overcomes to a certain extent one of the main difficulties in the upkeep and maintenance of water mains.

Pipe Jointing.—Cost and local ground conditions play an important point in settling the type of joint to be used, but the local custom of selecting spigot and socket joints in all horizontal positions (except for very high pressure pipes) and flanged joints where pipes are fixed vertically are so universally adopted that any description is not necessary. The author has used with success the turn and bored joint and can safely recommend same in bad situations, providing the pipes are laid in a straight line and an ordinary spigot and socket lead joint inserted every ninth or tenth pipe to allow for expansion in variable temperatures.

The laying of distributing pipes and mains presents very little difficulty, requiring engineering skill, especially in open ground. The only occasion where the work becomes intricate is in handling and laying large pipes in the centre of cities having their usual network of underground mains which have to be considered. On all pipe lines washout chambers should be built within reason, and especially at every depression, so that scouring out process may be easily accomplished, and an air valve placed at every high point to allow the imprisoned air to escape. Pipes should be laid true to line, grade, and on a good solid foundation, with particular care and attention paid for sufficient collar holes to facilitate and ensure the proper caulking of the joints. The mains, after being laid, should be subjected to proper tests and every length, when completed, should be closed by means of a blank socket, drilled and tapped to receive the connection pipe from the pressure pump, and a water pressure gauge showing the pressure registered in feet of water and lbs. per sq. in., with valves for safety, and lowering the pressure. An economical arrangement for testing the pipes is to commence laying, wherever practicable, from the source of supply, say reservoir, to the site of distribution.

All defects can be easily detected in the trench by careful observation and close attention to the indicator or pressure gauge, especially when pumping ceases. The most familiar defects are from weeping joints, split or cracked pipes and pinholes. The position of defect was marked on pipe, the pressure lowered, leakage made good, and the section re-tested. Weeping joints are generally easily put right by re-caulking. A split or cracked pipe requires more labor and attention, and in most cases its removal and replacement is the only satisfactory remedy. This can be accomplished in one of the following ways: (1) Burning out lead; (2) cutting out lead joint; 3) splitting off the socket; (4) replacing the whole pipe; (5) cutting the pipe, providing the defect is not too serious, the cracked portion taken out and a short piece cut to the required length inserted and jointed up by thimbles or sleeves. Pinholes in otherwise sound pipes may be drilled, tapped and plugged with brass or gun metal plugs.

Corrosion or rusting of pipes conveying water, both externally and internally, are serious items in the life of pipes supplying water as, although when properly coated with patent solution, corrosion is greatly reduced. But in time, unless properly inspected and removed, it takes effect, and although the danger of weakening the pipes is small, the real trouble lies in the contraction of the bore of the pipes, which diminishes their discharging capacity and reduces the working head.

Fire Service and Meters.—In conclusion the author would like to mention in connection with water supply for cities and towns—especially those in Canada—the absolute necessity of providing in all waterworks schemes an abundant supply to meet all the demands when called upon to save the destruction of lives and valuable property. This can only be successfully accomplished by the engineer making ample provision in the preparation of the scheme he is called upon to design; and by remedying the shocking waste of water in our cities through careless consumers who do not value or realize the expense a proper waterworks system entails to properly maintain out of the city's funds.

The advantage of having a powerful stream of water to be easily put into requisition to retard or prevent conflagration of buildings can be readily understood.

A little extra cost, if necessary arrangements are provided. in the original design of the waterworks, but may, as afterthoughts, prove an expensive addition.

The writer strongly recommends for use in case of fire, a surplus storage capacity over and above that required for general purposes, such storage to be of sufficient elevation to allow water being forced above the tops of the highest buildings in the cities, and also the arrangement and dimensions of the mains and distributing pipes to be capable of permitting the maximum fire supply when the demand for water for other purposes is at its greatest. In order to render a fire service efficient, hydrants should be placed at the most convenient and important points on the system to ensure a maximum efficiency. They should be in sufficient number and, above all, easily accessible, but not in danger of frost.

The available head at any given point on the system is calculated by deducting the loss of head due to friction in the pipes from the statical head at that point. It can plainly be seen that such results only hold good on a thoroughly watertight system, and it is imperative for hydrant purposes to maintain pressure. The laying and testing of mains should have the strictest attention of the engineer.

With reference to the prevention of waste, this important point must sooner or later be taken up seriously by Canadian cities, especially where pumping is the means of supply. The author strongly favors the insertion of water meters on the distribution services as an excellent prevention against extravagance and thoughtless excess in consumption. There are certainly advantages and disadvantages, the latter applying especially to the cheaper class of dwellings, for such a step might mean reversing the conditions as they exist at present and economy being substituted for undue extravagance at the expense of cleanliness and health. On sanitary grounds this would be a great objection, but at any rate the writer strongly advocates the adoption of meters for trade and manufacturing purposes, and also the serious consideration of meters on residential property with modifications to offset the disadvantage mentioned above. There are two types of meters, the "positive" and the "inferential." The former records by clock mechanism on similar principle to that of a cylinder and side valve of a high-pressure steam engine, water alternately filling and emptying a vessel of known capacity. The latter mechanically registers the flow by recording the revolutions made by a wheel with vanes or discs attached, on the principle of the water-wheel or turbine in a small chamber. The engineer should select the one which combines accurate results with varied flows, simple and easy repairing, and does not interfere with loss of head in supply pipe.

SUMMARY OF TESTS OF BOND BETWEEN CONCRETE AND STEEL.

(Concluded from last issue.)

(28) Pull-out tests made at early ages gave surprisingly high values of bond resistance. Plain bars embedded in 1:2:4 concrete and tested at 2 days did not show end slip of bar until a bond stress of 75 lbs. per sq. in. was developed. Bond resistance increases most rapidly with age during the first month. The richer mixes show a more rapid increase than the leaner ones. The tests on concrete at ages of over one year showed that the bond resistance of specimens stored in a damp place may be expected ultimately to reach a value as much as twice that developed at 60 days.

(29) The load-slip relation of leaner and richer mixes was similar to that for 1:2:4 concrete. For a wide range of mixes the bond resistance was nearly proportional to the amount of cement used. This relation did not obtain in a mix from which the coarse aggregate had been omitted.

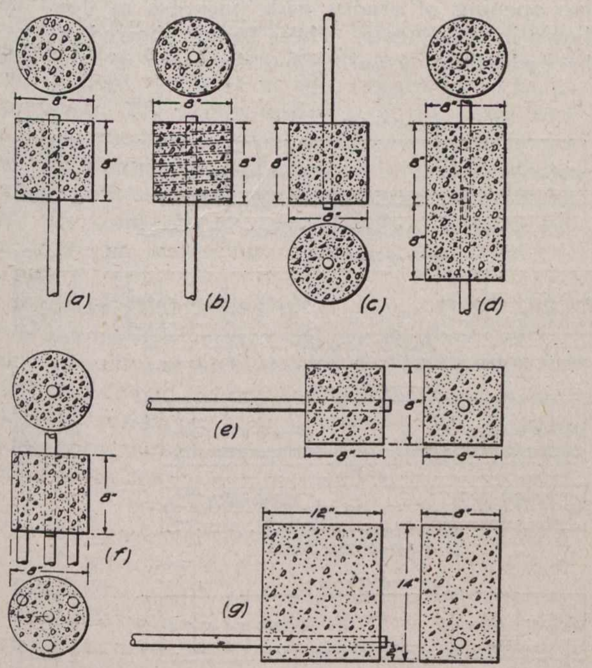

Fig. 1.—Types of Pull-out Specimens Used in the Tests.

(30) When the application of load was continued over a considerable period of time, or when the load was released and reapplied, the usual relation of slip of bar to bond resistance was considerably modified. The few tests which were made indicate that the bond stress corresponding to beginning of slip is the highest stress which can be maintained permanently or be reapplied indefinitely without failure of bond. The effect of continued and repeated load, impact, etc., may well be the subject of further experimental study.

(31) Little difference was found in the pull-out tests whether the load was distributed over the entire face of the block or over a narrow ring at the centre of the block or around the edge of the face of the block.

(32) Specimens molded in a horizontal position gave lower bond resistance than those molded in a vertical position; when settlement of the bar with the settlement of the concrete was entirely prevented, the bond resistance was reduced to about 60 per cent. of that found for similar specimens which were molded with the bars in a vertical position. Plain bars tested by being pulled in the same or the opposite direction from the settlement of the concrete during setting gave about the same bond resistance, but in the tests of certain deformed bars this was not true.

(33) The term "autogenous healing" is used to designate phenomena observed in pull-out tests and in compression tests of concrete cylinders in which the hardening of the concrete was interrupted by loading the specimen at early ages to its ultimate resistance. Up to an age of one year the bond resistance of specimens stored in damp sand was not affected by as many as four loadings at intervals during the period of storage up to the ultimate resistance. For specimens stored in air and tested in the same way, the bond resistance was less than for damp-sand storage, but the tests showed a steady increase in bond resistance with each loading up to three months. Specimens which had been stored in air for two months before the first test and in water thereafter showed a decrease in bond with each subsequent loading, although the bond resistance in the last test was fairly high. The presence of water apparently permits the continuation of the hydraulic action of the cement for several months after the mixing of the concrete.

(34) Bond resistance of plain bars is greatly increased if the concrete is caused to set under pressure. With a pressure of 100 lbs. per sq. in. on the fresh concrete for five days after molding, the maximum bond resistance was increased 92 per cent. over that of similar bars in concrete which had set without pressure. The greater density of the concrete and its more intimate contact with the bar seems to be responsible for the increased bond resistance. Light pressures gave an appreciable increase in bond resistance. With polished bars the effect of pressure was slight.

(35) As might have been expected, the compressive resistance of concrete setting under pressure was increased in much the same ratio as the bond resistance. At the age of 80 days the initial modulus of elasticity in compression for concrete which set under a pressure of 100 lbs. per sq. in. was about 37 per cent. higher and the compressive strength was increased by about 73 per per cent. over that of concrete which had set without pressure. The density of the concrete, as determined by the unit weights, was increased about 4 per cent. by a pressure of 100 lbs. per sq. in. on the fresh concrete. The increase in strength and density was relatively greater for the low than for the high pressure. A pressure continued for one day, or until the concrete had taken its final set and hardening had begun, seems to have produced the same effect in increasing the strength and elastic properties of the concrete as when the pressure was continued for a much longer period.

(36) Concrete cylinders tested in compression at age of 80 days after having been loaded to failure at 7 days gave compressive strengths nearly as high as those tested for the first time at the same age. Retests of cylinders which had set under pressure gave similar results.

(37) Beams of comparatively short span reinforced with bars of large size were used in order to develop high bond stresses and give bond failures. Most of the beams failed in bond; a few failed by a combination of bond and diagonal tension or by tension in the steel.

(38) The usual method of computing the bond stress in a reinforced concrete beam does not take account of all the phenomena of bond action. Slip of bar due to beam bond action and the presence of anti-stretch slip may be expected to greatly modify the distribution of bond stress over the length of the bar, and otherwise to affect resistance to beam bond stresses. However, the

nominal values for bond resistance, computed by the usual formula, form a useful basis for comparison in beams in which the dimensions and general make-up are similar.

(39) Slip of bar was a phenomena in all beam tests in which careful slip observations were made. These load-slip relations give important indications as to the bond stress developed at points along the length of the beam.

(40) Slip was first observed in the middle region of the span at loads producing a tensile stress in the steel of about 6,000 lbs. per sq. in. In this region the shear is zero, and hence beam bond action, as usually understood, is absent. As the load was increased, slip of bar progressed through the outer thirds toward the ends of the beam at a rate nearly proportional to the increase of load. After slip occurred at the ends, the outer thirds of the length of the bar moved toward the middle of the span relative to the adjacent concrete. Slip of bar was probably partly responsible for the opening of outer cracks, since slipping was observed in the outer thirds of the beams before the cracks became visible.

of the bar, was about 75 per cent. of that for similar beams reinforced with plain round bars of similar size.

(44) Beams reinforced with twisted square bars gave values at small slips about 85 per cent. of those found for plain rounds. At the maximum load, the bond-unit stress with the twisted bars was 90 per cent. of that with plain round bars of similar size.

(45) In the beams reinforced with 1⅛-in. corrugated rounds, slip of the end of the bar was observed at about the same bond stress as in the plain bars of comparable size. At an end slip of 0.001 in., the corrugated bars gave a bond resistance about 6 per cent. higher and at the maximum load, about 30 per cent. higher than the plain rounds.

(46) The beams in which the longitudinal reinforcement consisted of three or four bars smaller than those used in most of the tests gave bond stresses which, according to the usual method of computation, were about 70 per cent. of the stresses obtained in the beams reinforced with a single bar of large size. The progressive opening of cracks with increase in load was well shown in these tests. These beams showed cracks nearer the ends than usual. The distances of the outermost

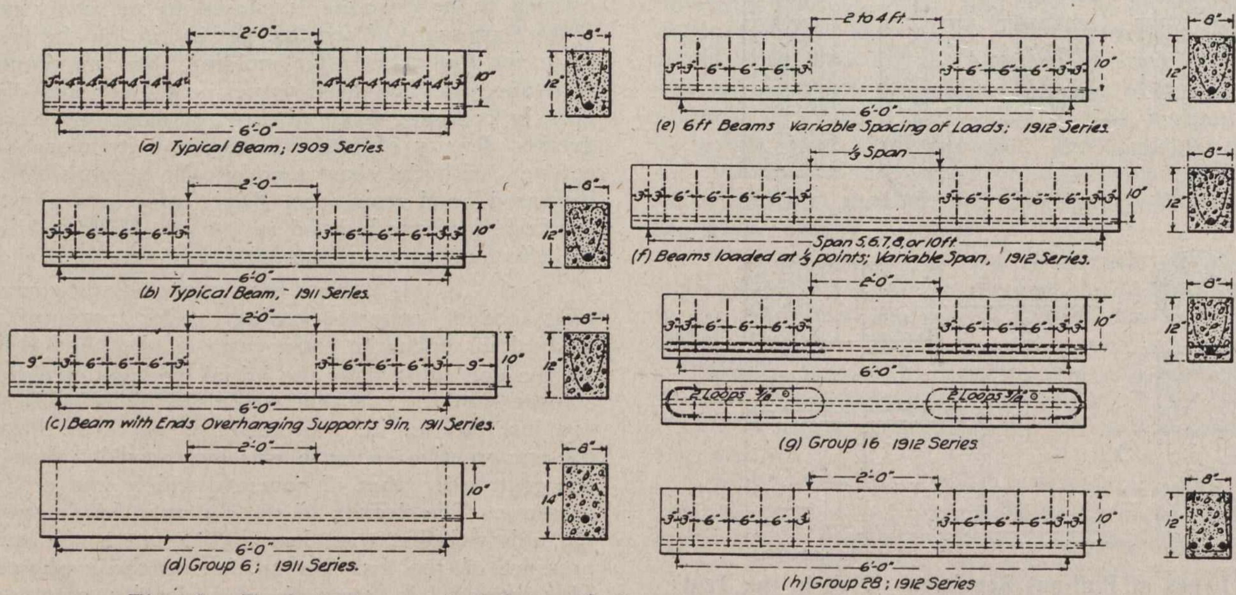

Fig. 2.—Typical Forms of Reinforced Concrete Beams Used in the Tests.

(41) The mean computed values for bond stresses in the 6-ft. beams in the series of 1911 and 1912 were as given below. All beams were of 1:2:4 concrete, tested at 2 to 8 months by loads applied at the one-third points of the span. Stresses are given in pounds per square inch.

	Number of tests.	First end slip of bar.	End slip of 0.001 in.	Max. bond stress.
1 and 1¼-in. plain round	28	245	340	375
¾-in. plain round	3	186	242	274
⅝-in. plain round	3	172	235	255
1-in. plain square	6	190	248	278
1-in. twisted square	3	222	289	337
1⅛-in. corrugated round..	9	251	360	488

(42) In the beams reinforced with plain bars end slip begins at 67 per cent. of the maximum bond resistance; for the corrugated rounds this ratio is 51 per cent., and for the twisted squares, 66 per cent.

(43) The bond unit resistance in beams reinforced with plain square bars, computed on the superficial area

cracks from the ends of the beams suggest that the unbroken length of embedment has an important bearing on the maximum loads which the beams may be expected to carry before failing by bond. It seems probable that the lower computed bond stresses in these tests are due to errors in the assumptions made as to the distribution of bond stress and not to actual differences of bond resistance in the bars of different size.

(47) The tests on beams with the loads placed in different positions with respect to the span gave little variation in bond resistance during the early stages of the tests. The maximum bond resistances increased rapidly as the load approached the supports. These tests indicate that the variation in the maximum bond stresses must be due to the presence of other than normal beam action.

(48) Nearly all the beams tested on span lengths of 7 to 10 ft. failed by tension in the steel and did not develop the maximum bond resistance, although high bond stresses were obtained. The bond stress corresponding to first end slip of bar did not vary much with the span length.

(49) The bond stresses developed in the beam tests indicate that with beams of the same cross-section the bond stresses are distributed in the same way during the early stages of the test in beams varying widely in span length and loading. During the later stages of the test, the distribution of bond stress seems to depend largely upon the conditions of stress in the concrete through the region of the span where beam bond stresses are high. The distribution of bond stresses in beams of different cross section apparently varies with the relative dimensions of the beam and the reinforcing bars.

(50) The use of auxiliary tensile reinforcement in the outer thirds of the beam served to modify the distribution of bond stress during the early stages of the test, but did not have any influence on the maximum bond resistance. While the auxiliary bars seemed to prevent the opening of outer cracks, the tests indicate that interior cracks which did not appear on the surface of the beam may have opened to an extent that permitted the same distribution of bond stress as was found in other tests.

(51) Increasing the thickness of the concrete below the reinforcing bars beyond the depth usually employed caused a very large increase in the resistance to bond and web stresses. The added stiffness of the beam and the increased flexural strength through the outer thirds of the span, prevented the formation of cracks in these regions. In the other beam tests such cracks were found to interrupt the continuity of bond action and to be an important factor in producing lower average bond resistances.

(52) Increasing the length of overhang of the ends of the beam beyond the support did not increase the resistance to web stresses as indicated by the opening of outer cracks, but it had an influence on the bond resistance. The bond resistance at first end slip was greater in the beams with the longer overhang. The maximum bond resistance was materially increased by the additional overhang.

(53) In the reinforced concrete beams it was found that very small amounts of slip at the ends of the bar represented critical conditions of bond stress. For beams failing in bond the load at an end slip of 0.001 in. was 89 per cent. to 94 per cent. of the maximum load found in beams reinforced with plain bars, and 79 per cent. of the maximum load for similar beams reinforced with corrugated bars. As soon as slip of bar became general, other conditions were introduced which soon caused the failure of the beam.

(54) The bond stresses developed in a reinforced concrete beam by a load applied as in these tests varies widely over the region in which beam bond stresses are present. High bond stresses are developed just outside the load points at comparatively low loads. The load which first developed a bond stress nearly equal to the maximum bond resistance in the region of beam bond stresses produced a stress near the support which was not more than about 15 to 40 per cent. of the maximum bond resistance. As the load is increased, the region of high bond stress is thrown nearer and nearer the support, and at the same time the bond stress over the region just outside the load point becomes steadily smaller. This indicates a piecemeal development of the maximum bond stress as the load is increased. The actual bond stresses in certain tests varied from less than one-half to more than twice the average bond resistance computed in the usual manner.

(55) Slip of bar in a reinforced concrete beam has a marked influence in increasing the centre deflection during the later stages of loading.

(56) The comparison of the bond stresses developed in beams and in pull-out specimens from the same materials is of interest. Such a comparison should be made for similar amounts of slip. In the pull-out tests the maximum bond resistance came at a slip of about 0.01 in. for plain bars. The mean bond resistance for the deformed bars tested was not materially different from that of the plain bars until a slip of about 0.01 in. was developed; with a continuation of slip the projections came into action and with much larger slip high bond stresses were developed. The beam tests showed that about 79 to 94 per cent. of the maximum bond resistance was being developed when the bar had slipped 0.001 in. at the free end; hence the bond stress developed at an end slip of 0.001 in. was used as a basis of the principal comparisons in the pull-out tests. However, it is recognized that, under certain conditions, the stresses developed at larger amounts of slip may have an important bearing on the effective bond resistance of the bar.

(57) The pull-out tests and beam tests gave nearly identical bond stresses for similar amounts of slip in many groups of tests, but it seems that this was the result of a certain accidental combination of dimensions in the two forms of specimens and did not indicate that the computed stresses in the beams were the correct stresses. However, it is believed that a properly designed pull-out test does give the correct value of bond resistance, and gives values which probably closely represent the bond stresses which actually exist in a beam or other member as slipping is produced from point to point along the bar. The relative position of the bar during molding may be expected to influence the values of bond resistance found in the tests.

(58) A properly made pull-out test on a specimen of correct design is a valuable aid in determining the bond resistance of reinforcing steel in concrete, if due consideration is given to the load-slip relation. The tensile stress in the bar should be kept well below the elastic limit. Best results will be obtained by using a relatively short embedment. An embedment of 8 diameters is recommended.

(59) A working bond stress equal to 4 per cent. of the compressive strength of the concrete tested in the form of 8 by 16-in. cylinders at the age of 28 days (equivalent to 80 lbs. per sq. in. in concrete having a compressive strength of 2,000 lbs. per sq. in.) is as high a stress as should be used. This stress is equivalent to about one-third that causing first slip of bar and one-fifth of the maximum bond resistance of plain round bars as determined from pull-out tests. The use of deformed bars of proper design may be expected to guard against local deficiencies in bond resistance due to poor workmanship, and their presence may properly be considered as an additional safeguard against ultimate failure by bond. However, it does not seem wise to place the working bond stress for deformed bars higher than that used for plain bars.

It is expected that Lord Kitchener, British Consul-General, will bring to England details of the plans for the barrage on the White Nile, about 37½ miles above Khartum, which are now being prepared by the Egyptian authorities. This great work is to form a necessary auxiliary to the Assiut barrage and the Esnch barrage, the present successful drainage schemes of Northern Egypt. It is expected that within 2 or 3 years, the area in the delta will have been reclaimed; and then the new barrage will be needed urgently. Commencement on the work will probably be postponed for another year, since the receipts from revenue will be low this year in Egypt; and the scheme is estimated to cost about $4,000,000,000.

TENDENCIES IN TRACK CONSTRUCTION.

At a joint meeting in Boston of the American Society of Mechanical Engineers, the Boston Society of Civil Engineers and the American Institute of Electrical Engineers, papers on present tendencies in railroad work were read and discussed. One of these, presented by A. B. Corthell, Chief Engineer, Boston and Maine R.R., had to do with track construction, and reviewed its development during the past half century or more. The Journal for July of the first-mentioned Society publishes an abstract of Mr. Corthell's paper, to which we are indebted for the following:—

The first steel rails made in America were rolled at Danville, Pa., in 1845. Other rollings were made in the same year by the Boston Iron Works, the Trenton Iron Works, the New England Iron Co. and the Phœnix Iron Co. The first Bessemer rail made in the United States was rolled in Chicago in May, 1865; the first Bessemer steel rails to be produced on a commercial order were rolled in Jamestown, in August, 1867. The introduction of the Bessemer process thoroughly revolutionized the art of rail manufacturing, and the ultimate effect on railway building and commercial development of our country can hardly be over-estimated.

Attempts were made about 1870 to roll a combination rail with steel head and iron web and base, but the rapid reduction in price of all-steel rails rendered this process of no economic value, for while steel rails in 1872 sold for $140 per ton, in 1882 the price had dropped to $35 per ton. This cheaper production made possible the heavier rail of recent years, also the larger locomotives, greater capacity cars, and correspondingly greater economy in railroad operations. It is interesting to note in this connection that there seems to have been a fixed relation between the weight of rail in pounds per yard and the weight of locomotives in tons, for when we had 60-lb. rails in general use, we had 60-ton locomotives, and with the 100-lb. rail came 100-ton locomotives; roughly speaking, in 70 years the weight of rails has increased 70 lb., or 1 pound per year.

Not many years ago the designing of rail sections had become a fad. Most engineers were called upon to get up a new standard design, and nearly all roads had their own standard sections. As a matter of record, the rail mills at one time had no less than 188 different patterns and 119 patterns of 37 different weights per yard. The situation was investigated by the American Society of Civil Engineers, and in 1893, after more than three years' deliberation, the Society reported upon standard sections for rail from 40 to 100 pounds, varying in weight in 5-lb. increments. This report was accepted by the Society and recommended to the railroads for adoption during the year 1901. Rails of the above type of sections constituted fully 75 per cent. of all the rails rolled in American mills.

In 1901 the report of the American Railway Association recommending the use of 33-ft. rails was adopted. In October, 1907, a preliminary report was submitted accompanied by two series of proposed standard rail sections, and in 1908 the report recommended types A and B. Since October, 1907, several mills have rolled rails substantially in accord with the new sections, both A and B, and it has been demonstrated that these sections can be finished in the mill at a lower temperature than the A.S.C.E. sections. A finer-grained and better-wearing rail should be secured. However, great care must be exercised in the mills to see that the rails are actually rolled at the lower temperature.

During the year 1913 there has been laid on the Boston and Maine Railroad 500 tons of 85-lb. frictionless rails in curves of 5½ deg. and over, with which it is hoped to lessen materially the flange wear on high rails, which on sharp curves is always considerable. Actual experiment shows that curve resistance is a great deal lessened by the use of this rail. The theory offered for the action of the so-called frictionless rail is based on the means that it offers the outer wheel on each axle to become dominant over the inner one, and the inner wheel to slide laterally to release the outer wheel flanges as they are forced against the outer rail. The outer wheel is traversing a greater distance through a curve than the inner one, but is making the same number of revolutions. On this account, a compensating slide of the outer wheel or a spin of the inner one must occur. The frictionless rail allows this necessary spin to occur at the inside rail.

No subject concerned with track appliances has been more discussed than that of the joint fastening. The evolution of joint fastenings has advanced through three stages: first, the chair which maintains the ends of the rail in alignment and serves as a bearing; second, the fish-plate, which afforded the rail some support under the head, but greatly improved the matter by stiffening the junction of the rails vertically; and third, the angle-bar, which combines the features of the fish-plate and flange, and effected a great improvement in both the vertical and horizontal stiffness of joint fastening; the plain angle-bar is very simple, easily applied and cheap in first cost.

The conditions, which bear some relation to the wear of splice-bars, are the extent of bearing surface and the hardness of the metal. In the new 85-lb. rails and smaller sizes, the question arises whether the plain angle-bar meets with the ideal requirements of the splice-bar in the two important respects, strength, and the wear in the immediate vicinity of the joint, which affects the close union of the parts. We know that angle-bars are not strong enough because they bend and take a permanent set in service, and occasionally one breaks. The supported joints which we have had in use are the Fisher, the Continuous and the Weber joints.

For bolts to fasten the joints to the rails, the most efficient are those having the so-called grip thread. This bolt is made of a soft steel, and the threads are cold-pressed in a manner to upset the metal so as to reduce the diameter of the bolt but slightly at the bottom of the thread. The threads are ratchet shape and under-cut 5 deg. on the bearing side. In the nut the bearing side of the thread is at right angles to the axis of the aperture, so that when it is screwed tight against the splice-bar the threads of the bolt give to the extent of which they are under-cut and the metal will be pushed completely to the outer recesses of the nut-threads, so as to hold the nut against turning off. The nut is square, with the corners chamfered next to the wearing surface, which gives an approximately circular bearing. On the bearing side the nut is recessed the depth of a thread and to a diameter somewhat larger than that of the threaded bolt, thus housing and protecting the many threads against injury by the chafing on the splice-bar.

The first tie-plates were used to prevent rails from cutting into and destroying the ties. Gradual development has added other features, such as the top shoulder, spike-hole, bottom claws and ribs, all tending to make the tie-plate not only a tie protection, but a more valuable rail-brace. Economy of material compels a minimum of weight consistent with strength, and one of the most important considerations is to obtain a tie-plate which will unite firmly with the tie; otherwise it will pound the tie and wear it under rail vibration and afford no lateral resistance to the spreading of the rails. As such a requirement cannot be had by a plate with a smooth underside, practically all tie-plates are now made with under projections in the shape of claws, which enter the wood crosswise of the grain, or of the flanged or rib type, which enter the wood longitudinally with the grain. In the former case the lateral displacement of the plate is resisted by an abutment against an end section of the fibres. The standard Boston and Maine tie-plate has four flanges which enter the grain of the tie longitudinally, running the width of the plate. The latest tie-plate shows the two longitudinal flanges and two smaller transverse flanges on the bottom, a heavier shoulder and a better portioning of material.

Wooden ties have been almost universally used by the railroads of this country, and are still used as best practice. Steel ties and ties of concrete construction have been made, and are used to some extent with varying success. For wooden ties, the hardwood tie of oak, chestnut and hard pine are used mostly for main line traffic, and the softer woods, such as cedar, for branch lines of light service. The standard Boston and Maine tie is 6 in. thick by 8 in. wide and 8 ft. long. The average life of a chestnut tie is seven ears, and hard pine ties eleven years. The life of a tie can be lengthened by the use of tie-plates and preservatives.

There is indication of no radical change in the present track materials or methods in the immediate future. The rail may be heavier and more spikes, tie-plates and braces added, but the general design will be the same. The changes in turnouts and yards will be most marked; longer switch leads, wider spacings of track, heavier rail and more careful maintenance are already necessary in a great number of our yards, due to the increased loads in power and rolling stock. In the Pennsylvania Terminal in New York City we find part of the tracks laid on stone ballast and some part on a solid concrete base, with creosoted ties bedded therein and anchored by bolts to the concrete.

CONSTRUCTIONAL ACTIVITY IN MONTREAL.

The iron and steel trade in Montreal is greatly interested in the various reports given out from time to time by the big producing firms of the country. The statement of the Nova Scotia Steel and Coal Company for June was regarded as quite encouraging, compared either with a year ago or with the previous month. The figures are as follows, outputs being given in tons:—

	June, 1913.	May, 1914.	June, 1914.
Ore mined	47,200	38,903	44,000
Ore shipped	......	70,129	60,000
Coal mined	67,088	69,349	71,600
Coal shipped	......	61,677	90,000
Pig iron	7,220	......	7,000
Steel ingots	7,147	6,668	10,600
Ingots rolled	......	......	3,770

From the above figures it would seem that the company maintained its output in a satisfactory manner. The activity was evidently greater in June than in May, and the statement is made that the output of steel ingots reached a new high record. This result is somewhat surprising when it is remembered that it was generally considered that outputs of steel companies were very low in June. The trade is looking forward to other companies' reports.

It has developed during the past week or so that orders for structural steel have been running light and it is stated that the principal bridge companies are laying off draughtsmen and other employees. On the other hand, it is asserted that the cement companies are more actively employed, and that they are doing a better business than a year ago. The same is asserted for the brick factories, while other lines of industry send in somewhat similar reports.

INITIAL STRESSES IN STRUCTURAL STEEL.

THE importance of investigation into the subject of initial stresses in steel for structural purposes has assumed a prominence in recent years that is well justified when one is led to observe the low elastic limits, evidently due to initial stress, which tests made on rolled steel shapes have disclosed. Engineers who have noted fractures in fabricated steel work that are indicative of disproportioned stresses and strains are generally inclined to lay the blame upon internal initial stresses. The subject has been well treated by Mr. J. R. Worcester before the Boston Society of Civil Engineers, and from his paper we reproduce the following interesting discussion:—

For many years it has been recognized that where steel is heated locally by forging, there is likely to be produced a region between the heated and unheated portion where the metal is brittle and can be broken by a blow or shock. A striking example of this defect came under the writer's observation recently in the case of some 2-in. diameter steel truss-rods which had been upset. One of these, in unloading from a wagon, had its upset end broken off with a granular fracture. On testing the other rods, by striking the ends with a sledge, it was found that several broke in the same manner, while the remainder could not be broken. A chemical analysis of the fractured ends gave the following result, which is consistent with good metal: carbon, 0.408 per cent.; sulphur, 0.045 per cent.; phosphorous, 0.065 per cent.; and manganese, 0.38 per cent. Although not so certainly established as in other cases, it is quite probable that this fracture was induced by internal stresses caused by the local heating.

A similar defect in I-bars was noticed, soon after the introduction of steel into their manufacture, and led to the universal adoption of annealing furnaces long enough to anneal the whole bar at one time after forging. This practice has recently been proved by Mr. A. H. Emery to be of no benefit, as far as can be determined by tensile tests, as it decreases both the elastic limit and the ultimate strength, while no decrease in strength appears to follow from the local heat treatment, under direct tension as applied in the testing machine. It does not necessarily follow, however, that the annealing may not be a desirable precaution as a safeguard against shock.

Admitting, then, the prevalence of initial stresses, it is interesting to consider their origin with a view to guarding against them where possible. When their origin is in some form of heat treatment it is generally possible to overcome them by annealing, although this may not be the only or the best method. When they are confined to members used in direct tension they may not be of serious import, because, on the application of stress to the member, the effect is to increase the initial tensile stresses already existing, reducing or neutralizing internal compressive stresses. As the applied load increases, a point is soon reached where the fibres carrying most of the tension reach the elastic limit and begin to stretch, after which a redistribution of stress occurs, spreading the stress equally over all the fibres.

A familiar example of this is in the case of a copper wire which may be coiled or crooked with internal stresses. We all know how, if such a wire is stretched beyond the elastic limit all crooks immediately disappear. So with a steel member, if in tension it is in stable equilibrium, and a minute stretch can usually occur without harm to the structure.

With compression members, however, the case is radically different. In these, the first applied load, if conflicting initial stresses exist, tends to throw the whole member out of alignment. It is in unstable equilibrium, and the more it bows the greater the danger.

One of the causes of initial stresses is cold straightening of metal before assembling. Cold straightening is, in reality, cold bending, and the following investigation is an attempt to determine the limits of internal stress which may be produced by cold bending.

It is well known that material as it comes from the hot bed is almost always more or less out of line, and that in order to straighten it the most effective and simple method, and the one generally used, is to bend the member, in the direction opposite to the initial curvature, enough so that when it springs back under its elasticity its alignment will be true. The effect of this bending beyond the straight line and allowing the elasticity to bring it back to the correct point is to strain the outer fibres on each side beyond the elastic limit. The elastic recovery reverses the stress in the extreme fibres which have been overstrained, and leaves a condition of stress within the section similar to that shown in Fig. 1.

This means that starting from one edge of the section we find at first a tensile fibre stress extending a certain distance in constantly decreasing intensity until it reaches a point of zero stress, or a secondary neutral axis, beyond which the stress becomes compressive, increasing to a maximum at a point, the distance of which from the outer fibre is the same as that which limited

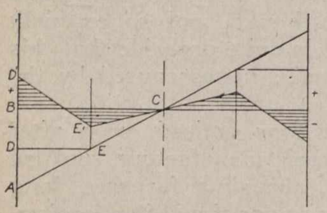

Fig. 1.

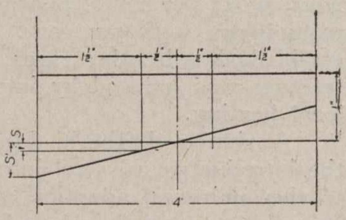

Fig. 2.

the field of metal stressed above the elastic limit by the bending. From this point the compressive stress diminishes to the axis of the section, beyond which it becomes tensile again, increasing to a certain point from which it again decreases, changing to compression at another secondary neutral axis and increasing in compressive stress until the opposite extreme fibre is reached.

By considering Fig. 1 we see at once that with a symmetrical rectangular section we have two fields of tensile stresses and two fields of compressive stresses represented by triangles, and we find that certain assumption may be made with regard to these fields which serve to fix their amounts. In the first place, considering the effect of the bending, if the material was not strained beyond the elastic limit, the stresses on each side of the neutral axis would have been represented by a single triangle ABC. If, however, the stress AB is greater than the elastic limit of the metal, this triangle would be truncated by a line DE parallel to the cross-section and distant from it an amount represented by the elastic limit of the material. When the bending stress is relieved, the line DEC assumes a new position $D'E'C$, the distance from D to D' and from E to E' being proportional to the distance from the neutral axis. This proportionality is one of the determining elements. Another is the fact that the total tensile stress multiplied by the distance of its centre of gravity from the neutral axis must equal the compressive stress multiplied by its axial distance. This is a necessary condition of equilibrium.

Lest it be argued that the line DEC does not agree with the stress-strain diagram, it should be borne in mind that in considering the elastic distortion we are dealing with an extremely minute deformation. Between adjacent planes of cross-section it is really infinitesimal, and any finite stretch would be so small as to produce practically no increment in stress. This consideration is valueless on account of its imaginary character. We might, therefore, without invalidating the argument, assume that we are considering the angle between two planes of cross-section separated by a finite distance, as, for instance, 1 in.

Suppose, for example, a bar 4 x 1 in. bent edgewise, to a radius of curvature such that one-quarter of its width along the neutral axis is still elastic and the remainder, on each side, is overstrained. Referring to Fig. 2, the stretch S, at the limit of the elastic portion,

will be about $\frac{30{,}000}{30{,}000{,}000} = 0.001$ in. The stretch of the extreme fibre S' will be

$$0.001 \times \frac{1.5}{0.5} = 0.003 \text{ in.}$$

The assumption is that the elongation of $S' - S$ will not cause an appreciable increase in stress after the metal has reached the elastic limit, i.e.,

$$S' - S = 0.003 - 0.001 = 0.002 = 0.2 \text{ per cent.}$$

If the gain in strength between the elastic limit and the ultimate strength is accompanied, as it frequently is, by a stretch of 30 per cent., and we assume that the rate is proportional, this stretch of 0.02 per cent. would mean an increase of stress of about

$$\frac{0.02}{30.0} \times 30{,}000 = 200 \text{ lbs.}$$

But, the characteristic of stress-strain diagrams is that there is a sudden yielding accompanied by a very considerable stretch, with no increase in stress; in fact, there may be a slight decrease in stress. It is evident that, in assuming any sharp angle in the diagram, there is a slight error, as the corner should be rounded. It would be more exact to say that the lines assumed are tangent to the curves, but the effect of this rounding may be disregarded without invalidating the theory. Referring to Fig. 3, the above assumptions may be expressed algebraically as follows:—

$$x_1 + x_2 = a \quad \ldots \quad (1)$$

$$\frac{x_1}{y} = \frac{x_2}{z} \quad \ldots \quad (2)$$

$$y + e = u + e - w \quad \ldots \quad (3)$$

$$\frac{v + e}{e} = \frac{a + b}{b} \quad \ldots \quad (4)$$

$$\frac{w}{z} = \frac{a + b}{b} \quad \ldots \quad (5)$$

$$\frac{x_1}{2} y \left(a + b - \frac{x_1}{3}\right) = (b + x_2) \frac{z}{2} \left(\frac{b + x_2 + b}{2}\right) \frac{2}{3} \quad \ldots \quad (6)$$

Solving these equations, we obtain these values for y and z in terms of a, b and e:—

$$y = \frac{2(a + b)^2}{a(a + 2b)} e; \quad z = \frac{2(a + b)^3}{a^2(2a + 3b)} e.$$

Letting $a + b = 1$, we find that the equation for y, assuming a as a variable, is a parabola with its vertex at the neutral axis of the section with a value of ½e, the parabola passing through the origin (see Fig. 4). This equation is

$$y = \left(a - \frac{a^2}{2}\right)e, \text{ or } 2a - a^2 = \frac{2y}{e}.$$

Expressing this result in words, it amounts to this: If a rectangular bar is bent so that it has any permanent set, the internal maximum fibre stress may be obtained if we know to how great a depth the outside portion of the section has been stressed beyond the elastic limit. The amount of this internal stress can never exceed one-half the elastic limit, and between o and ½e it varies according to the abscissas of a parabola of which the axis is the neutral axis of the section.

We may determine the depth to which fibres are stressed beyond the elastic limit if we know the radius of curvature and the thickness or depth of the section. We know from mechanics that $\frac{1}{r} = \frac{e}{Ed}$, in which r is the radius of curvature, e is the elastic limit of the material, E is the modulus of elasticity, and d, the distance

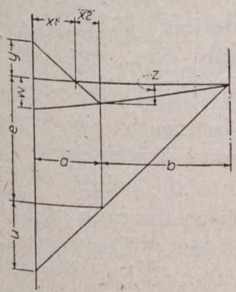

Fig. 3.

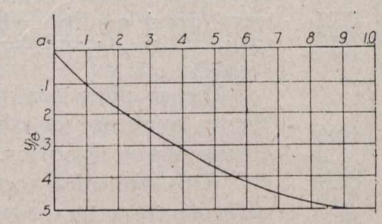

Fig. 4.

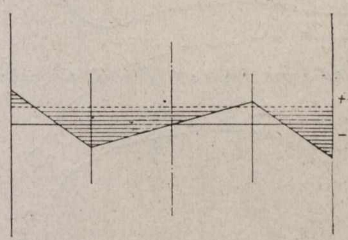

Fig. 5.

from the neutral axis to the extreme fibre. Using 30,000 for e and 30,000,000 for E, this formula becomes, $r = 1{,}000\,d$. In other words, the distance from the neutral axis to the fibre which is strained just to the elastic limit will be 1/1,000 of the radius of curvature; hence, if we know approximately the radius of curvature, we can tell at once what part of the thickness of the section is not overstressed, and, subtracting this from one-half the total thickness, we can find a. Taking a practical example of this, we should obtain the following results:—

Assume a bar 3 x 1 in. to be somewhat crooked edgeways and to be straightened in a press. Let us assume that in straightening it is curved to a radius of 12 ft., a very moderate assumption.

The width of metal not overstressed would be

$$\frac{12 \times 12}{1{,}000} = 0.144 \text{ in. each side of the neutral axis.}$$

a would, therefore, $= 1.5 - 0.144 = 1.356$ in.
or, on the basis of $a + b = 1$, $a = \frac{1.356 \times 1}{1.5} = 0.9$.

From the diagram, Fig. 4, we find that under these conditions, y, the initial fibre stress amounts to $0.495\,e$, tension on one edge and compression on the other, or approximately 15,000 pds. per sq. in. This means that in a bar which is quite straight and wholly "innocent" in appearance there may exist a compressive stress along one edge of 15,000 lbs. per square inch, while along the opposite edge is a tensile fibre stress of an equal amount; in other words, an inherent tendency to bend out of line on the least "provocation." This condition cannot be detected by any of the usual methods of inspection, but might be suspected if we knew its history.

It will be noted that the above analysis applies only to a rectangular section. In the case of an irregular section, such as an I-beam, it is evident that if the bending is in the plane of the web, a lesser stress in the extreme fibre will produce equilibrium on account of the decreased area of the section in the parts nearer the neutral axis. On the other hand, if the bending is at right angles to the web, the converse is true, and the extreme fibre stress will be greater proportionally, and may easily approach nearer to the elastic limit. The same is true of a bar with a circular cross-section.

Let us now consider the practical effect of these internal stresses. Referring again to Fig. 1, we see that if we apply an axial stress to a member which is already subjected to this condition of internal stress the effect will be to produce a condition as shown by Fig. 5. In this case we see at once that the areas of stress will be unbalanced, so far as the rotating moment is concerned. The effect of this unbalanced condition will be to produce a tendency to spring out of line. If the axial stress is in tension, this tendency is offset by the axial stress itself, and even in case the extreme fibre stress exceeds the elastic limit, a slight yielding of these fibres soon distributes the stress more uniformly, and so no serious results can occur. But if the axial stress is compressive, the tendency to spring is very serious and immediately throws the strut out of equilibrium, so that the bad effect of the internal fibre stress is accentuated. If the elastic limit is passed, the buckling may even continue to the point of failure.

Mr. Worcester did not enlarge upon applications of the above theoretical considerations. His paper, nevertheless, shows the tremendous importance of eliminating cold straightening so far as possible from the shop treatment of metal which goes into compression members.

On June 19, Kingston dock, Glasgow, Scotland, was completely destroyed by fire, entailing damage to the extent of $1,250,000. The original cost of the dock was $750,000; but extensive alterations and improvements have been in progress during the past year. The collapse of the quays carried with them huge cranes, thousands of feet of iron girders, and rooms of sheds. In addition four schooners, laden with seal oil and other products, were destroyed.

According to M. Hans, Dempwolff, of the staff of the Prussian Government Railway, and a recent visitor to Canada, a large scheme is now being considered in Berlin for electrifying the suburban and interurban railway systems, which are run on elevated tracks as in a number of the leading American cities. The cost of the work would involve the expenditure of something like 150,000,000 marks ($30,000,000.) The German Government, he remarked, was experimenting with electric locomotives, and was trying out Deisel engines on its lines.

WHARNCLIFFE HIGHWAY BRIDGE, LONDON, ONT.

By F. M. Brickenden.

THE city of London, Ont., with its population of 48,000 is situated on the River Thomas. The location with respect to the main river, and particularly its north and south branches, is such as to require a considerable number of highway bridges to convey the traffic from one part of the city to another. A new bridge, to be called the Wharncliffe highway bridge, is at present under construction. It is the first to cross the main river, and is one the need of which has been felt for a number of years, particularly since the annexations of London South and London West, both of which were until a comparatively short time ago municipalities outside the city. The site is shown in Fig. 1. It is about ¼ of a mile downstream from the forks of the north and south branches of the Thames.

Before its municipal development London West was a low-lying piece of land, and has had to be protected on the river side by a breakwater. London South, however, in the vicinity of the bridge site, is on an elevated strip of land. Further, owing to the directions of the currents produced by the junction of the two branches of the river, and taking into consideration the ice brought down in the spring, assisted by the fact that the two centre lines of the Wharncliffe roads in London South and London West do not coincide, it was found necessary to design the bridge on a skew. The angle of the substructure was accordingly set out at 113° 29′.

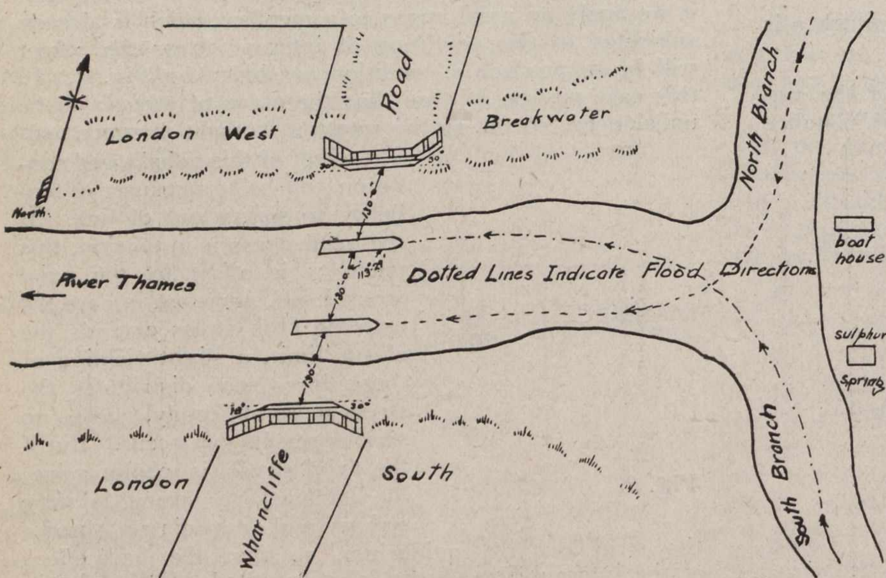

Fig. 1—Location and Layout of the Wharncliffe Highway Bridge, London, Ont.

Excavation.—Underlying the bridge site is what is known locally as London South white clay. This clay is used is another locality for the purpose of making white brick. It is of such a nature as to puddle readily upon mixing with water. Because of this considerable difficulty was experienced in excavating for the south abutment of the new bridge. Clay slid into the excavation on several occasions. The work was started in August last, and the contractors adopted the pick-and-shovel method of removing the clay to a depth of approximately 20 ft. For a part of the time a team and slush scraper were employed. As the clay developed drying cracks on exposure, blasting with black powder was found ineffective when tried.

When the excavation had attained a depth of about 10 ft. a 10-in. cast-iron water main was encountered, necessitating, of course, its removal and relocation to one side of the bridge site. The pipe was found to lie directly under the entire length of the proposed bridge. As the river reaches a depth of about 3½ feet, at low water mark, a special arrangement was necessary to place this 300 feet of water main on the bottom without breaking the caulking. The entire river length was accordingly placed on the bents, each about 9 ft. high. It was then hung from the threaded rods by a wire. The rods projected through the top of the bents and then held by wrenches. To lower, 28 men were placed, one at each bent, and on a given signal, each gave his wrench one turn, thus lowering the whole as a unit. The excavation of the south pier footing was simply sheet-piled, and on reaching the bottom 30 hardwood piles were driven and cut 9 in. up from the bottom level. For the north pier a wood caisson was built, and between the outer and inner parts the intention was to fill with earth and sink to a firm foundation. This was done, but considerable trouble was experienced because of the pressure of the water on the outside of the cofferdam, forcing through the light filling material used in sinking the framework. This was pumped out by means of a centrifugal pump, driven by an 8 h.p. gasoline engine (a small, power diaphragm pump was used in pumping out the other pier excavation). This foundation, although of very good quality white clay, was filled with 30 piles driven to refusal.

The 30 piles for the south pier were driven by means of a horse-driven capstan, lifting about 1,500 lbs. to the height of 18 ft. This operation took an average of 111 seconds from blow to blow. For driving the remaining 30 piles a small hoisting apparatus was connected up with the 8 h.p. engine by a chain, and from the hoisting-drum a cable was fastened directly to the hammer of the pile-driver, so that after raising the hammer it fell, drawing out the cable and ready to start again. The operation of this method consumed at best but 11 secs. from blow to blow, although much time was wasted, in that the leaders used were unsuitable.

The excavation for the north abutment was through the West London breakwater, about half being taken out by means of team and scraper, and the remainder removed by shovel.

Both the abutments were laid out with the short centre line as the line of the bridge-seat on the coping. This line located, the counterfort walls were at right angles to the face, so that the angle turned assisted in the layout of these forms. The angles of the wings were all turned with the transit. The layout of the south pier was simply by direct means, but, as the north pier was entirely in the water at its lowest, the method used for it was different. A wire was stretched along centre line, with the centre point marked equidistant from pier and abutment. The skew was first set by measuring along a triangle applied to the caisson and scaled off the plan. This was checked by measuring along sides of a parallelogram from shore, and later having a stage placed on the cofferdam when the angles were checked with the instrument.

It may be mentioned that the city of London controls the height of the water in the Thames by their waterworks dam, four miles down the river; so that the level of the water was reduced 3 ft. during pier construction last fall and this spring.

Reinforcing.—The building of the counterforted abutments gave rise to several noteworthy conditions. For example, when the supporting walls or counterforts were being formed, they were battered up from a base-slab, which was 12 ft. in thickness at the bottom, and 57 ft. or 52 ft., according to the abutment, to the top of the ballast wall. At the points of wings and main wall the counterforts are 2 ft. thick, double the width of the remainder. The inclined reinforcing was hooked at the rear of the base reinforcing and ran up 3 in. from the back of the counterforts to the top. The rods used were of three sizes, ½ in., ¾ in., ⅞ in., all square. The base-slab was reinforced by having rods ¾ in. diam. placed across the whole of it, parallel at right angles to the face. To this all the vertical and inclined rods were hooked, thus making the base serve as an anchorage against overturning. Besides the inclined ⅞-in. rods in the counterforts, 3 pairs of ½-in. rods of different lengths were placed vertically, so that they ran to the inclined rods of the latter, and were hooked to them.

Parallel to the base and running 3 in. from the face, other rods were placed up to the bridge-seat. In front of these and in a vertical position were other ½-in. rods, to which were hooked the small, horizontal rods of the counterforts. In the bridge-seat were a number of short ¾-in. rods, laid parallel to the short diameter, and wired to these were several long ½-in. rods, horizontal and parallel to the face.

The ballast wall was reinforced by having short ¾-in. vertical rods, attached to long ½-in. horizontal rods, along the entire length of the wall.

Piers.—The north foundation forms and cofferdamming were radically different from that of the south, which consisted of a sheet piling only. The portion nearest the shore was shallow and easily kept dry, even when the river rose somewhat above the lowest water level obtainable for the work. A ring of clay around the excavation formed a barrier to the water. The seepage water was, as stated, removed by a power diaphragm pump.

The forms for the pier were of simple construction except the nosing. It had to be specially detailed to suit the cut-waters, which were only rolled as regular right angles rather than specials. The detail showed the nosing running back from the ⅞-in. cut-water angle at right angles to nose-point, or at a tangent to its curvature. The amount of curvature did not change, but the tangent or right angle of the nosing varied for each 2 ft. of rise, leaving the joint between tangent and curve vertical and imperceptible. This nosing then required a special form, built up in shapes roughly represented by the letter "U." These fitted over the cut-water angle, holding the form boards in place. The boards were placed vertically, being strongly wired to the 2 x 6-in. uprights which were outside the U-shaped forms.

Generally, the scantling was placed 18 in. apart and wired every 2 ft. with No. 9 galvanized, medium hardness, wire. All forms were oiled with a heavy oil, acting as a filler. Openings of any size were plugged with hard soap.

The north pier occasioned the placing of two cofferdams, one being heavily timbered, with an outer and inner shell. The caisson principle was tried, and this first floated into position and sunk. It required a great deal of filling, followed by many days of pumping before the excavation could be started in the river bottom.

Owing to the lateness of the season there was much uncertainty about the weather. The floods, carrying ice, washed the whole cofferdam away, filling in the work done during the early winter with river muck and debris. To get this in shape again for concrete a different method was used to get the old excavation cased in again, and watertight. A framework was built of 2 x 6's,

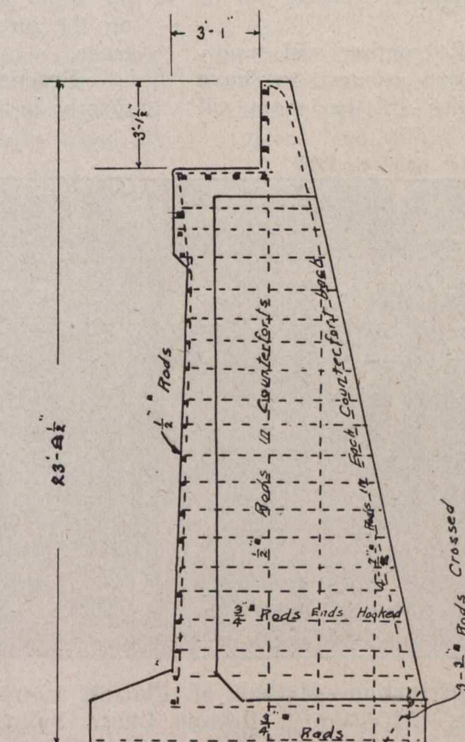

Fig. 2—Section of Counterfort, North Abutment.

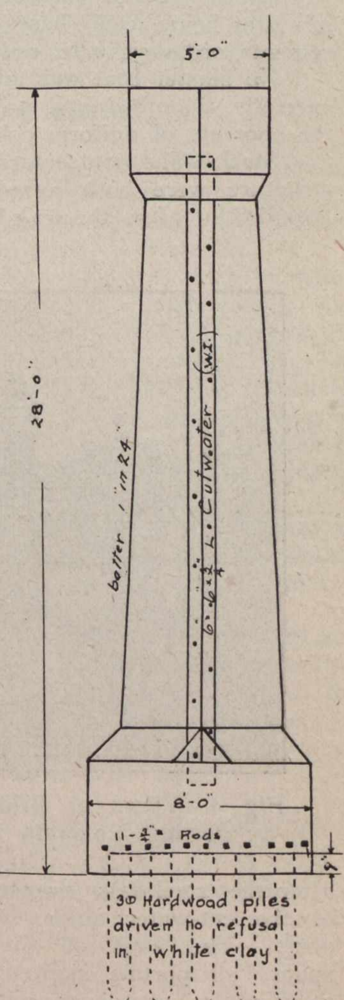

Fig. 3—Typical Pier Section.

covered with tongue-and-grooved lumber, and stiffened by corner-braces. The lower, or downstream end, was left uncovered. The boarding reached a height of 6 ft., this being sufficient for the depth of the low water then in the river. This was floated over the piling, which had survived the onslaught of the ice, and was sunk into place, the open end being sheet-piled. Around the whole cofferdam were placed sand-bags and loose sand. This, with the aid of the centrifugal pump, reduced the water again to a workable depth.

Concrete.—The abutments were concreted first. The base of each was concreted for 6 in. and the base reinforcing placed. Concrete was then poured to the level of the base of the vertical walls and top of base plinth. The upper reinforcing was held in position by a framework.

After placing the forms the concrete was placed to the coping to allow the bridge-seat reinforcing to go in place, then continued to the base of ballast wall. The long, horizontal rods were then wired to the short uprights, allowing the completion of the remainder.

The placing of concrete for the pier was more difficult. The abutments only required a few lengths of semi-circular troughs, with a bent-end and a small hopper, while the piers required about 150 ft. of troughs, supported on a trestle-work that carried also a plank walk, for workmen.

When the piers rose to about one-third their height, a series of stages were erected at the form end of the chutes, upon which were men who passed up the concrete from one level to another to its position in the pier.

About 700 cu. yds. of concrete were placed, three mixtures being used; bases in water, 1 :6 + 25 per cent. cement; ordinary, 1 :6; nosing piers, 1 :4.

No plaster coat was added to the faces. They were carefully smoothed and a wash applied, making all of the concrete of uniform color.

Steel.—The contractors for substructure and superstructure were both agreeable upon request to leave the drilling for the rag-bolt holes in the seats till after the steel was erected and centred; the rag-bolts of the cut-water angles were concreted in with the nose. The steel of the bridge consists of three spans, all portals skewed, and the downstream side carrying a cantilever sidewalk (bracketted at each panel joint). The bridge spans are through rivetted trusses. The steel is being erected by what might be called the "Armstrong" method. An idea of the size of the connections and splices may be formed by the fact that the whole structure will require about 7,000 rivets, field-driven. The structure rests on three types of bearings. The south, or expansion end, is of a roller type, resting on a bearing-plate and supporting a slotted plate from the connection of end-post and bottom chord; five rollers to each corner, each roller of 3½ in. diam. The plate is held in position by 2-in. rag-bolts. The bearing-plates all rest on sheet lead, which serves to prevent the action of the free alkali on the steel plates. The bearing casting for the fixed end is a casting over 4½ in. thick. It is partly cast hollow in pockets on the underside, thus reducing the weight, but retaining sufficient strength to resist compression. It rests directly on bearing-plate. The third style used is a direct bearing-plate on abutment fixed directly off the connection-plate.

The portals are skewed, so that the ends of the end floor beams are sawn at flanges. The angle connections are special to suit 113° 29′ and 66° 31′. The remaining floor beams are placed at right angles to the bottom chord. The stringers are also cut obliquely at the ends of span to fit over the skew beams. The bottom laterals are angles, with legs 2½ x 2½ in., except end panels, which are 4 x 4 in., being rivetted at the centre. The top struts are built of 7-in. channel, edged to web of 6-in. channel, and hung from their connection at the top panel joint. The top laterals are 2½ x 2½ in.

The knee braces attached to the posts and top struts give greater stability from wind stresses which are rather excessive in this valley. The portals are arranged with angles and channels on edge, giving a neat appearance. The sidewalk consists of cantilever brackets having 10-in. I-beam stringer in centre for floor, a channel with angle on upper flange for curb of walk. A latticed railing is on the outside of the walk and inside of the upstream truss.

Erection.—The erection presented a few difficulties, mostly in the placing of the falsework in water—the height which the falsework bents had to rise above the water, compared to the depth of the river, gave no great amount of buoyancy in raising the heavy timberwork. Besides this difficulty, the falsework of the first span, though placed through the ice, had two bents washed out by a flood following the first week of erection, requiring much patience to take lines across the river by means of a rowboat and fasten them there. The erection of the second span being entirely over water, was more noteworthy. The depth of the water was about 7 ft., so that it was not convenient to work a "gin" pole in the river. A built-up "stiff-leg" was placed, projecting over the southerly pier and stayed back to the steel already erected. A bent was built up on shore of 12 x 12 in. material, braced, capped and towed under a hook operated by a pair of "three" blocks swinging from the "stiff-leg." To the end of the loose, threaded line a second pair of "three" blocks was attached and laid out by which the gang of men were able to raise the bent to almost its proper position.

By moving this arrangement to the top of each bent as raised, the eight bents required were speedily placed in their position and braced.

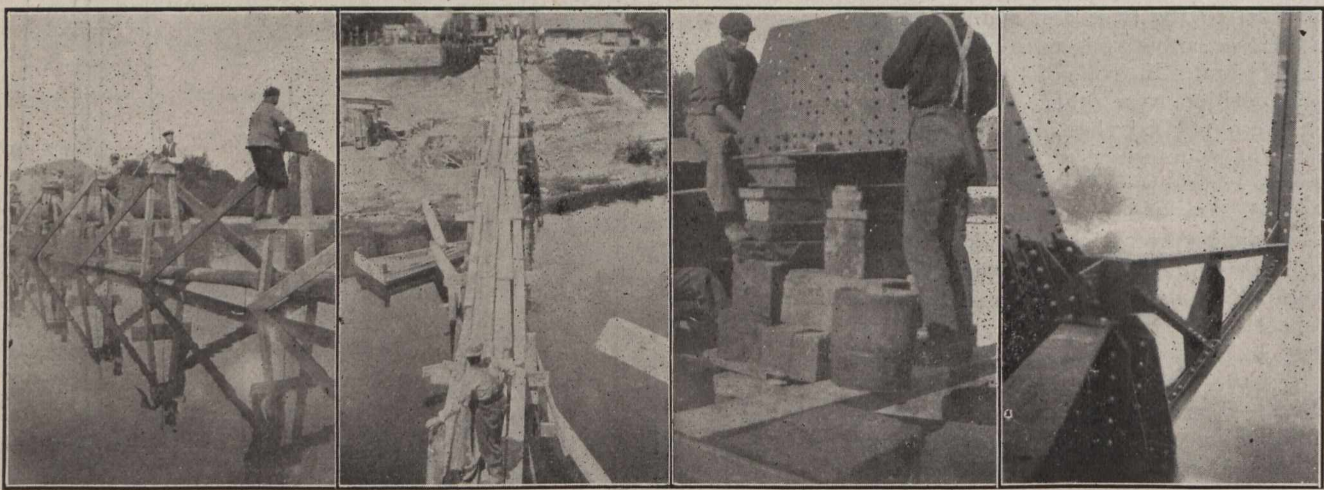

Fig. 4—Views of Bridge Construction—Method of Placing 10-in. C.I. Water Main—Chute for Transporting Concrete to Piers—Rivetting a Bottom Chord Splice—Sidewalk Cantilever Adjustment.

The bottom of the river at this point is gravel over clay, and, as there was no settlement of the falsework, the bents upon being placed were connected longitudinally over the caps by 3 and 4-in. lumber.

In the centre a runway was made by laying down the I-beam upon the centre pair of longitudinal timbers. This double runway was to accommodate the movement of the steel from the storage to its position in the span by means of a cart with a tongue and looped axle arrangement, whereby the steel when attached to the axle could be raised clear of the ground, and by balancing its connection was easily shoved along by a few men. The longest of the heavier top chord sections, 40 ft. in length, was moved this way.

After placing 12-in. I-beams to serve as a runway over the falsework, the bottom chord was next brought into position and rivetted at its splices. This was necessary and expedient at this juncture, because the chord was built up of 2 angles, edge to edge, at end panels, and 4 angles in the middle of the span to take up the greater stresses existing there. This in position, with its vertical connections at the panels, and 2 diagonals, hangers, or end-posts, would have made the splice rivets impossible to drive when all was erected.

Floor System.—The floor system consists of 24-in. I-beams across at all panel joints, about 20 ft. apart, except the ends of the span, or skew beams, which are both 20-in. I's. Because of the oblique angle, 66° 31′, the permanent connection between the lugs and end-post base is made with turned bolts and lock washers. Centered over this beam system, 12-in. I-beams form the stringers, those over the end skew-beams having to be

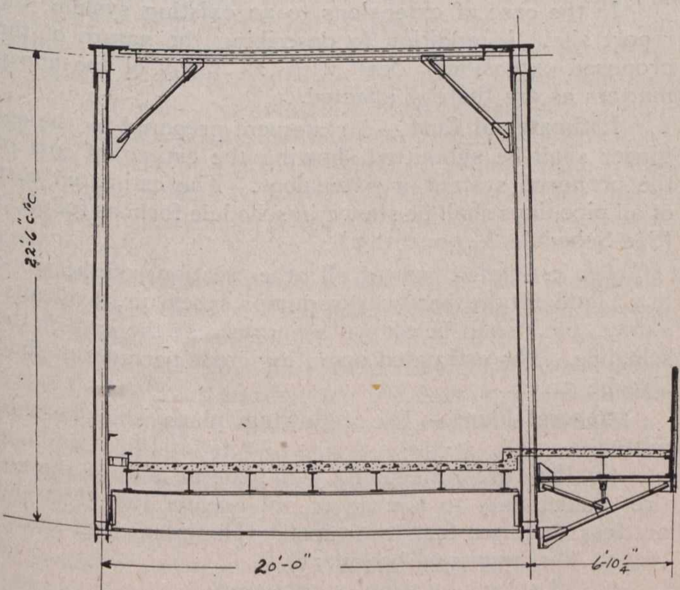

Fig. 5—Section of Bridge at a Centre Panel Joint.

of special length. The stringer opposite to the sidewalk, on the upstream side of span, has a channel on its top flange which forms, with back inwards, the curb of the roadway.

On raising the end-posts, a block and line were attached to the top of the hanger by which the end-post could be hauled into position and bolted at the top to the hanger and to the lower chord at its low end, resting on the bearing-plate of pier. A "traveller" was made of 8 x 19-in. timbers, with upright legs, high enough so that the projecting ends of cap-piece, suspending two sets of blocks, would not come in contact with the top chord or verticals till they were above their position, and could easily be lowered into the position when ready for bolting. The same block method was used as in raising the bents, these pieces were raised by the double three blocks, requiring about 10 men on the line.

Posts or verticals were all placed previous to raising the diagonals. To place them the "traveller" rig had to be shifted by means of pinch-bars, the bottom of both uprights moving along the same thickness of plank. The guy-ropes were held by fastening them around the various

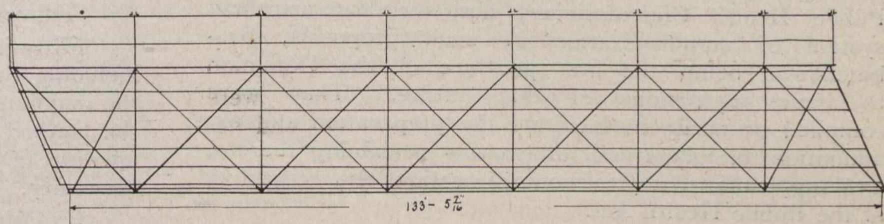

Fig. 6—Floor System.

erected members. In the case of the most distant top chord, these ropes had to be taken across the river and there fastened to a post on the breakwater. This method applied also to the extreme posts of the span.

Laterals and Struts.—Rivetting was done by hand, the outfit consisting of 3 men and a heater. Top and bottom laterals, knee-braces and struts of portals are erected as the rivet gang reaches the connection. The same can be said of the cantilever sidewalk brackets, which were held out by block and line, then set in position by the coupling-up pins and bolted. Only such of these as were needed to resist wind stresses were placed at the time of erection on the falsework.

The floor is to be reinforced concrete roadway of 18 ft. clear width, and the sidewalk 6 ft. The total length of floor is 409 ft. 1 in. This conforms to the, Ontario Government specifications for a 15-ton bridge.

The substructure was built by the firm of Bain and Ross. The steel is supplied and erected by the Hamilton Bridge Company. The work is being done under the direction of Mr. W. N. Ashplant, the City Engineer.

Among large buildings at present under construction at Saskatoon, Sask., are a large store and office block, a cold storage warehouse, a distributing warehouse for the Prairie City Oil Company, a business block for the Canadian Townsite Properties, Limited, a new sedimentation basin, and an additional students' residence at the University of Saskatchewan. The interior storage elevator for the Dominion Government also is expected to be completed for this season's harvest. While the new bridge across the Saskatchewan River at Twenty-Fifth Street, being erected at a cost of about $500,000, is to be completed by the fall of 1915.

The Taku Railway and Navigation Company has filed in the land office at Juneau, Alaska, right-of-way and terminal plans for the proposed railway line up the Taku River from a point on Taku Inlet to the Canadian-Alaska boundary. About two miles of side cut rock work on the lower end is encountered. The line for the most part is tangent and free of short or sharp curves. It follows the flat in the valley almost the entire distance and tranverses three tracts of good timber. The grade is said to be considerably less than 1 per cent. The terminal is located on the south side of Taku Inlet just opposite Windom Glacier, which has been dead for some time. The Inlet is narrow at this point and not over 40 feet in depth; and the feasibility of bridging the canal at this point and continuing the line to Juneau has been discussed and is believed by engineers to be practicable. F. J. Wetrick, of Wilhelm and Wetrick, who made the survey for the American line of the railway, is now making a preliminary survey on the Canadian side to Atlin.

SASKATCHEWAN REGULATIONS GOVERNING WATER AND SEWERAGE WORKS.

THE Bureau of Public Health of the Province of Saskatchewan has just issued some important regulations governing the preparation and submission of plans and information with reference to the installation and extension of waterworks, sewerage and sewage disposal systems. These were prepared by Mr. R. H. Murray, A.M.Inst.C.E., engineer to the Bureau, under the direction of Dr. M. M. Seymour. Public Health Commissioner, and form an excellent system of standardization of such proposals, Saskatchewan being the first province of the Dominion to issue regulations of this nature. They were compiled primarily to facilitate the preparation and examination of plans and information pertaining thereto, and supplement the provisions of sections 21, 22 and 23 of the Public Health Act.

Respecting the plans and data required for provisional and final certificates, the following constitute the requirements of the Commissioner for Waterworks systems and extensions: Engineer's report; estimates of cost; general plans; analysis of proposed source of supply, and affidavit covering the same; detail drawings, specifications and affidavits covering both.

For sewerage and sewage disposal systems and extensions the same items apply with the exception of the analysis of proposed source of water supply. In the case of storm sewerage systems and extensions, affidavits will be required to cover general plans, also.

If the information and data submitted with reference to any proposed system or extensions are satisfactory and complete in the first instance, the Commissioner may issue his provisional and final certificates together.

Waterworks Systems and Extensions.—A report shall be submitted prepared by the engineer acting for the municipality giving information under the following heads, in so far as such are applicable to the proposed works:—

Population—The present population and the rate of increase during each of the past five years; available data justifying a future increase of population and probable increase in next ten years.

Maximum population which system will provide for if fully developed.

Population provided for by proposed system.

Area and topography of municipality.

Estimated water consumption per capita per twenty-four hours.

Alternative sources of supply (if any) and if investigated.

Source of supply recommended.

Present available supply of water and how determined.

Estimated supply of water under full development.

Quality of water.

Any present sources of pollution of source of supply.

Measures recommended to prevent future pollution of source of supply.

Nature and construction of intake or collection works.

Wells—The number, depth, size and construction of wells; the nature of the ground through which they are sunk; the construction of collecting galleries.

Watercourses—Minimum dry weather flow, approximate watershed area. If continuous winter flow; reduction in available supply during winter.

Catchment areas—Approximate area of watershed, character of watershed surface with reference to probable run-off, population, arable and stock farming and all available data having reference to rainfall.

Natural lakes—Approximate area of watershed, average depth of lake, area of lake, if overflow is continuous, approximate annual overflow, nature of watershed relative to population, arable and stock farming, and all available data having reference to rainfall. Population on shores winter and summer. If surface is used for traffic in winter.

The capacities and character of pumps; duplication of machinery and energy.

Sedimentation—The size and construction of basins.

Filtration—A full description of the proposed plant including the type of filter, the nature of filtering media, the maximum rate of operation of each unit of the plant; the method of cleansing filtering media; the nature and amount of coagulent or disinfectant estimated to be necessary.

Provision for future extensions.

Provision for measuring and recording amount of water supplied to municipality.

Provision made for storage and capacity of reservoir, stand pipes, etc.

Size of force or gravity mains.

Distribution system, especially with reference to cut-off valves, sluice valves, hydrants and through circulation; minimum size of pipes and minimum depth of trench; character of subsoil.

Pressure available for domestic and fire purposes.

Particulars as to any provision made for turning water other than the domestic supply into mains for fire protection purposes.

Provision to be made for inspection of construction.

Control and operation.

In the case of extensions to an existing system, the report shall, in addition to describing the nature of the proposed extensions, deal with as many of the above matters as are thereby affected.

Estimates of Cost.—A statement prepared by the engineer shall be submitted showing the estimated cost of the proposed system or extensions. The estimated cost of all pipe lines shall be shown in schedule form as follows: (See Schedule A, page 183.)

The estimated cost of all other sections of the work (e.g., land, intake, headworks, pumps, reservoir, hydrants, valves, etc.) shall be shown separately at the end of the schedule. The estimated operating costs per annum shall also be given.

General Plans.—The following plans shall be submitted:—

(*a*) Plan of Municipality.—A general plan of the entire municipality to a scale of not greater than 200 and not less than 500 feet to an inch. This plan shall show:

1. The municipal boundary.
2. All streets existing or proposed.
3. The approximate location of all habitable buildings not served with water at the date of application.
4. The surface elevations at all existing or proposed street intersections.
5. The elevation of the highest point in the municipality.
6. The location and size of all existing and proposed water mains.
7. The location of stand pipes, valves, hydrants and all appurtenances.

(*b*) Location Plan.—A plan to a suitable scale showing the site of the municipality, any outside territory affected, and the following:

1. The source of supply, intakes, wells, filters, pumps, reservoir and any special features.

2. The route of gravity or force mains.

3. The following elevations shall be shown: (*a*) The low, mean and high water levels of the watercourse or lake at the intake. (*b*) The highest known flood elevation of the watercourse or lake. (*c*) Surface and water elevations at standpipes, reservoirs and principal points in the system. (*d*) Surface elevations showing any irregularities of route with reference to the hydraulic gradient.

This plan is not required in the case of extensions to a distribution system.

Analysis of Proposed Source of Supply.—A sample of the water from the proposed source of supply shall be taken not more than six weeks previous to the date on which application is made to the Commissioner of Public Health for his certificate of approval. The sample shall be subjected to chemical analysis and bacteriological examination. A copy of the result of such analysis shall be submitted to the Commissioner of Public Health. The commissioner may require any number of samples extending over a stated period of time, if necessary.

The above shall also apply to each application for approval of extensions to an existing system, the sample being taken from the existing mains, should there be no proposed change in the source of supply.

An affidavit shall accompany any report of water analysis stating that the report of analysis and examination submitted is a true copy of the report supplied by the analyst, and that the water analyzed and examined was taken from the proposed source of supply. The date on which any sample was taken shall also be stated in the affidavit.

Detail Drawings.—Detail drawings shall be submitted to a scale which shall clearly indicate the design of all sections of the proposed system or extensions, including all wells, collecting galleries, intakes, filters, settling basins, reservoirs, conduits, standpipes, siphons, blow-offs, pumps, machinery buildings and other appurtenances.

In the case of a gravity supply, a profile of the route of the pipe line shall be submitted, showing surface elevations, the hydraulic gradient of pipe line, and air and sluice valves.

Specifications.—Specifications shall be submitted covering all work to be undertaken in the proposed system or extensions.

All detail drawings and specifications shall be accompanied by affidavits stating that the drawings and specifications so submitted are those to be used and followed in the construction of the proposed system or extensions.

Sanitary Sewerage Systems and Extensions.—A report shall be submitted, prepared by the engineer acting for the municipality, giving information under the following heads, in so far as such are applicable to the proposed works:—

1. Population—The present population and the rate of increase during each of the past five years. Available data justifying a future increase of population and probable increase in next ten years.

2. Maximum population which system will provide for if fully developed.

3. Population provided for by proposed works.

4. Area and topography of municipality.

5. Water consumption in municipality per capita per 24 hours.

6. The character of the sewage (with reference to surface and roof water, trade wastes, etc.)

7. Estimated maximum rate of flow expressed in gallons per hour.

8. Estimated normal dry weather flow expressed in gallons per 24 hours.

9. The general nature of subsoil throughout the municipality.

10. Precautions to prevent infiltration in water-bearing strata.

11. The minimum grades of all sizes of sewers.

12. Provision made for flushing and the intervals of flushing.

13. Character of pipes and joints to be used.

14. Type of sewage lifts to be adopted.

15. The maximum distance between manholes.

16. Ventilation of system.

17. Storm overflow discharges and any points of discharge other than the sewage disposal plant.

18. The areas of the municipality which cannot drain by gravity into proposed system. The proposed method of providing drainage for such areas in the future.

19. Provision to be made for inspection of construction.

20. Control and operation.

In the case of extensions to an existing system, the report shall, in addition to describing the nature of the proposed extensions, deal with as many of the above matters as are thereby affected.

Estimates of Cost.—A statement, prepared by the engineer, shall be submitted showing the estimated cost of the proposed system or extensions. The estimated cost of all pipe lines shall be shown in schedule form as follows: (See Schedule B, page 183.)

The estimated cost of all other sections of the work (e.g , manholes, inspection chambers, sewage lifts, etc.) shall be shown separately at the end of the schedule. The estimated cost of operation per annum shall also be given.

General Plans.—The following plans shall be submitted:—

(*a*) Plan of Municipality.—A general plan of the entire municipality to a scale of not greater than 200 and not less than 500 feet to an inch, this plan shall show:

1. Municipal boundary.

2. All streets existing or proposed.

3. The approximate location of all habitable buildings not connected with sewers at the date of application.

4. The surface elevations of all existing or proposed street intersections.

5. The location, size and direction of flow of all existing and proposed sewers.

6. The location of all existing and proposed manholes, lampholes, flush tanks, sewage lifts, sewer outlets, overflows and other appurtenances.

7. Any areas from which it is proposed to pump the sewage; these should be indicated by light coloring or shading.

(*b*) Contour Plan.—A contour plan of the municipality to a suitable scale.

Detail Drawings.—Detail drawings shall be submitted to a scale which shall clearly indicate the design of all parts of the proposed system or extensions, including all manholes, lampholes, flushtanks, siphons, pump wells, inspection chambers, buildings, iron work, machinery and other appurtenances. Transverse sections of all sewers over 24 inches in diameter shall be submitted.

Profiles of all sanitary sewers shall be submitted. The following scales are suggested for general adoption:

Vertical	10 feet to an inch
	or 20 feet to an inch
Horizontal	80 feet to an inch
	or 100 feet to an inch
	or 200 feet to an inch

Profiles shall show all manholes, lampholes, flush-tanks, siphons, river and railway crossings, the size and grade of sewers, the elevation of sewer invert and ground surface at all manholes and changes of grade; elevation of river or stream beds crossing the line of sewers.

Specifications shall be submitted covering all work to be undertaken in the proposed system or extensions.

All detail drawings and specifications shall be accompanied by affidavits stating that the drawings and specifications so submitted are those to be used and followed in the construction of the proposed system or extensions.

Storm Sewerage Systems and Extensions.—A report shall be submitted, prepared by the engineer acting for the municipality, giving information under the following heads, in so far as such are applicable to the proposed works:—

1. Precipitation—All available data having reference to the maximum precipitation in the municipality, particularly with reference to short periods of heavy precipitation.
2. Nature of ground surface—The areas paved and unpaved, and in the case of the latter, the degree of porosity of the ground.
3. The area and mean slope of each district draining to a trunk sewer or point of discharge.
4. The data and assumptions upon which the computation of the size of sewers is based.
5. The estimated maximum volume of flow (in cubic feet per second) at each point of discharge, and the estimated increase when the system is fully developed.
6. The character of pipes and joints to be used.
7. The nature of coating (if any) to be used on the outside of pipes.
8. Any connections which it is proposed to make with the sewers, other than for surface water.
9. Provision to be made for inspection of construction.
10. Control and operation.

In the case of extensions to an existing system, the report shall, in addition to describing the nature of the proposed extensions, deal with as many of the above matters as are thereby affected.

Estimates of Cost.—A statement, prepared by the engineer, shall be submitted showing the estimated cost of the proposed system or extensions. The estimated cost of all pipe lines shall be shown in schedule form as follows: (See Schedule C, page 183.)

The estimated cost of all other sections of the work (e.g., manholes, catch basins, etc.) shall be shown separately at the end of the schedule. The estimated cost of operation per annum shall also be given.

General Plans.—A general plan of the entire municipality shall be submitted to a scale of not greater than 200 and not less than 500 feet to an inch; this plan shall show:

1. Municipal boundary.
2. All streets existing or proposed.
3. The surface elevations of all existing and proposed street intersections.
4. The location, size and direction of flow of all existing and proposed storm sewers.
5. The location of all existing and proposed manholes, catch basins and connections to the storm sewerage system, also all points of discharge and natural watercourses.
6. The extent of street paving.
7. The areas draining to each trunk sewer or point of discharge (these may be shown by distinctive coloring or shading).

Detail Drawings.—Detail drawings shall be submitted to a scale which shall clearly indicate the design of all parts of the proposed system or extensions, including all manholes, catch basins, valves, etc. Transverse sections of all sewers over twenty-four inches in diameter shall be submitted.

Specifications.—Specifications shall be submitted covering all work to be undertaken in the proposed system or extensions.

All plans, general plans, detail drawings and specifications shall be accompanied by affidavits stating that the plans, drawings and specifications so submitted are those to be used and followed in the construction of the proposed system or extensions.

Sewage Disposal Works and Extensions.—A report shall be submitted, prepared by the engineer acting for the municipality, giving information under the following heads, in so far as such are applicable to the proposed works:—

1. Population—The present population and the rate of increase during each of the past five years. Available data justifying an increase of population and probable increase in next ten years.
2. Maximum population which system will provide for if fully developed.
3. Population provided for by proposed works.
4. Water consumption in municipality per capita per 24 hours.
5. Estimated maximum rate of flow of sewage arriving at works, expressed in gallons per hour.
6. Estimated daily flow of sewage arriving at works.
7. The number of connections already made to sanitary sewerage system.
8. The character of the sewage (with reference to surface and roof water, trade wastes, etc.). Any excess over domestic sewage shall be estimated in gallons per hour under: (*a*) Roof water; (*b*) surface water; (*c*) infiltration water; (*d*) trade waste water.

   In the case of trade discharge the character of the waste shall be given.
9. The distance and location in relation to the works of any wells or underground source of water supply. In the case of the final effluent discharging into a watercourse or lake, it shall be stated whether such watercourse or lake is, or eventually becomes, a source of domestic supply or is used for watering cattle.
10. Minimum volume and velocity of stream or river into which the final effluent will be discharged.
11. Area of land acquired for sewage disposal purposes.
12. Character of subsoil through which sewerage system is laid, with particular reference to the presence of sand.
13. Character of treatment processes to be adopted.
14. Type of sewage lifts to be used at sewage disposal works.
15. Provision for dealing with sewage in the case of failure of sewage lifts.
16. Type of screens and provision for the removal of screenings.
17. Preliminary precipitation of mineral particles—velocity of flow in detritus tanks.
18. Type of sedimentation tanks, capacity and velocity of flow in tanks, inclination of base of tanks.
19. Method of drying sludge and final disposal.
20. Capacity of dosing or siphon chamber.
21. Type of filter beds; character, area and depth of filter media. Rate of filtration. Method of distribution over surface of media.
22. Retention of humus.
23. Disinfection of effluent.

24. Provision against frost.

25. Location of bypasses.

26. Provision for measuring and recording flow of sewage.

27. Future extensions.

28. Provision for grading, surfacing, etc.

29. Control and operation.

In the case of extensions to existing works, the report shall, in addition to describing the nature of the proposed extensions, deal with as many of the above matters as are thereby affected.

Estimates of Costs.—A statement prepared by the engineer shall be submitted showing the estimated cost of the proposed works or extensions. The estimated cost of each section of the work (e.g., land, pipe lines, sedimentation tanks, filter beds, mechanical appliances, buildings, etc.) shall be shown separately. The estimated cost of operation per annum shall also be given.

General Plans.—The following plans shall be submitted:—

(*a*) Location Plan.—A location plan to a suitable scale showing:

1. The location of the works in relation to the site of municipality.

2. The route of the main outfall sewer.

3. The layout of the various units of the sewage disposal works, including all drains, pipe lines, etc.

4. The area of land to be utilized for sewage disposal purposes.

5. The lake, watercourse or subsoil, into which the final effluent will be discharged.

6. Elevation of high, mean and low water in watercourse or lake.

7. Highest known flood elevation of watercourse.

8. Elevation of effluent drain at point of discharge and of outfall sewer at entrance to works.

(*b*) Sketch Plans.—Preliminary sketch plans shall be submitted to a scale which shall indicate the type and capacity of the various tanks, filter beds, etc.

The principal elevations and measurements of the various parts of the works shall be indicated on these plans. Such preliminary plans shall give sufficient information to enable the commissioner to arrive at a conclusion as to the efficiency of the proposed processes of treatment and design, and structural details may be omitted therefrom. Sketch plans need not be submitted if the detailed drawings required by section (4) accompany the application.

Detail drawings shall be submitted to a scale which shall clearly indicate the design of all parts of the proposed works. Longitudinal and transverse sections of all tanks and filters shall be shown.

Profiles of all pipe lines and drains shall be submitted drawn to a suitable scale.

Specifications shall be submitted covering all work to be undertaken in the proposed system or extensions.

All detail drawings and specifications shall be accompanied by affidavits stating that the drawings and specifications so submitted are those to be used and followed in the construction of the proposed works or extensions.

In addition, there is a set of general regulations applicable to the above respecting dimensions of drawings, samples, records, etc. A penalty is attached covering violation of the regulations.

Schedule A.

Location	Between	Size of pipe	Character of pipe whether steel, cast iron, etc.	No. of existing buildings requiring water	Average depth of trench	Length in feet	Cost of pipe laid complete per lin. foot	Total cost

Schedule B.

Location	Between	Size of pipe	Character of pipe whether fire-clay, concrete, etc.	No. of existing buildings requiring drainage	Average depth of trench	Grade %	Length in feet	Cost of pipe laid complete per lin. foot	Total cost

Schedule C.

Location	Between	Size of pipe	Character of pipe whether fire-clay, concrete, etc.	Average depth of trench	Grade %	Length in feet	Cost of pipe laid complete per lin. foot	Total cost

Some of the building contracts of engineering importance which are at present in progress at London, Ont., are the construction of two public schools at about $50,000 each, the Dominion Savings building at $200,000, the Wellington Street Methodist Sunday School, $12,000, a building for the Ford Motor Company to be used for the assemblage of cars and for a warehouse, $50,000; the reconstruction of the Princess Avenue School, $30,000; a large addition to the St. Joseph's hospital, and the reconstruction and enlargement of the Collegiate Institute

Recent reports state that the Illinois Central Railway has placed orders with the Pressed Steel Car Co., the American Car and Foundry Co., and the Standard Steel Car Co., for 2,000 cars, which will make a total of 5,000 purchased by that railway in June. Other orders reported are one from the Wabash Railway Company for 60 locomotives from the American Locomotive Co., and one from the Seaboard Company to the same engine builders for 30 locomotives. June has proven to be a considerably better month for steel manufacturers than did either April or May.

REPAIRING LEAKS IN AN 8-INCH MAIN 40 FEET UNDER WATER.

About two years ago a water main of 8-inch cast iron pipe with lead caulked flexible joints was laid across Galveston Channel, which is the waterway lying along the wharf front at Galveston. It extended from the wharf front, where it connected with the city water main, to the pile dike on the north side of the channel, a distance of 1,400 feet, thence was laid in a shallow trench along the dredge spoil bank. The portion across the channel was laid in a trench 40 to 100 feet wide, dredged to a depth of 41 feet across the entire width of the channel, which was then about 30 feet deep for 500 feet of its 1,400 feet width. Cast iron pipe was used with flexible joints of the ordinary ball-and-socket type, which allowed a deviation from a straight line of about 12°. The ordinary lead caulking was used with yarn packing. The greater portion of the line was laid by caulking up and lowering one pipe at a time, the line of pipe being carried on an inclined trough from the barge nearby to the bottom of the trench and the barge being fleeted forward as the line lengthened. Soon after the laying was completed and water had been turned into the main, leaks began to develop due partly to the method of laying and to the uneven character of the bottom, which caused such a great deflection at each joint that the lead caulking was squeezed out, and to the fact that the line

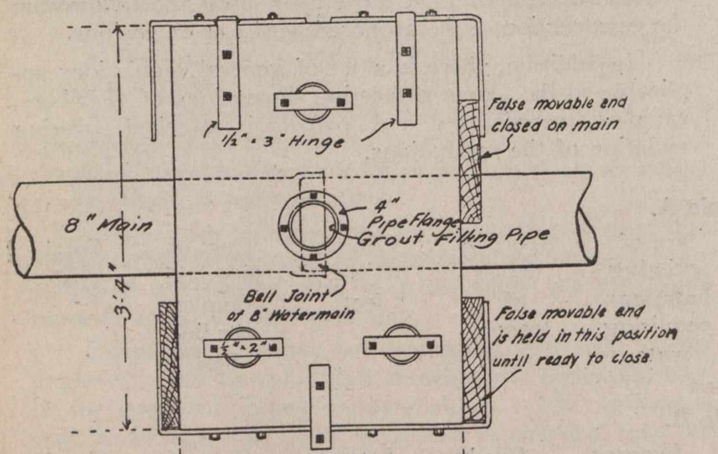

Fig. 1.—Top Plan of Form Used in Repairing Water Main.

was caught by a ship's anchor before completion. Efforts were made to recaulk the leaky joints by the aid of a diver, but these efforts were unsuccessful as it was found the pipe was deflected at these points to such an extent as to close the caulking space on two sides. After considerable thought and investigation, a method of stopping the leaks was evolved which has proven entirely successful. The method is described by Mr. N. T. Blackburn in the July-August issue of Professional Memoirs of the U.S. Army Corps of Engineers, as follows:

The method was simply the placing of a wooden box form around each joint and filling this form with neat Portland cement grout. Each form was supported on three 3-inch pipes, 14 feet long, driven down through holes in the forms into the underlying clay so as to form a pile foundation and prevent any settlement at the joint due to extra weight.

The form, shown in detail in the accompanying drawings, is 3 feet 4 inches square by 2 feet deep, hinged at one edge so that it opens diagonally. In the top is a 4-inch hole with a pipe flange for connecting a 4-inch pipe extending to the top of the water, and through which the grout was poured from the barge into the form. To the front of the lower half was fastened a chain which helped close it and held it closed afterwards, one link being let over a hook attached to the upper half and screwed up until form closed tight. In either side, where main passed through, a square hole 18 inches by 18 inches was left in order that the form would close no matter if the joining pipes were at their greatest possible angle, either vertical or horizontal. A movable section consisting of two pieces, 2 inches by 12 inches by 3 feet, each cut out in the form of an 8-inch semi-circle to take half of the main, closed around the main, tightly covering and at the same time entirely closing the 18-inch square holes. These false ends work outside of the form and are held in position by iron straps under which they can move in any direction against its side. Three holes large enough to take 3-inch pipe were bored through the top and bottom of the form for the piling. They were so bored that two piles would go on one side of the main and one on the other. After the pipe piles were driven down through the form flush with the top, a one-half inch iron plate was laid over the top of the pile and bolted to the form to carry the weight until the cement set around the piling.

The trench in which the main lies had been partly refilled with clay thrown into it by a dredge and by the natural deposit of sand, mud, and silt carried in by the cross currents and tides. This filling was partly removed with a 20-inch suction dredge, but fear of disturbing the main kept the dredge from working closer than about 4 feet of its top. After the dredging was done the water was cut off the main, air pressure put on and leaks were located by the air bubbles coming to the surface of the

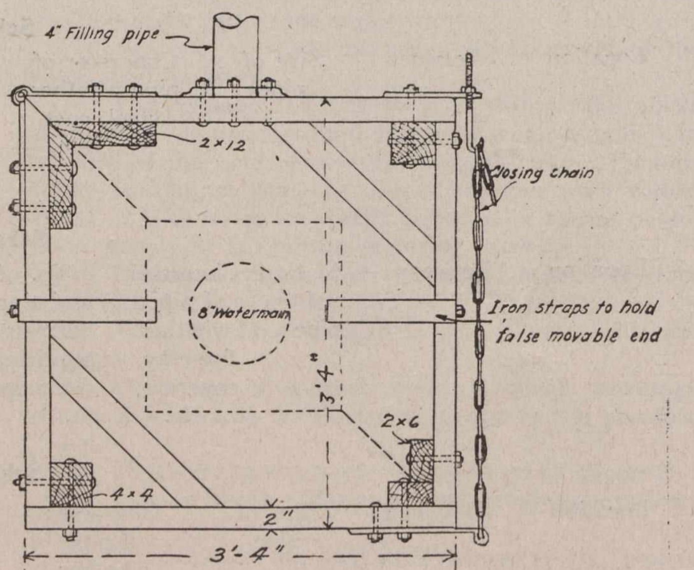

Fig. 2.—Side Elevation of Form.

water. All leaks of any consequence were marked by dropping a weight into the hole blown through the mud over a leak by the air and carrying a line ashore from the weight. It was not safe to use buoys for marking the leaks, as they were liable to be carried away by ships and all leaks had to be located and marked before repairs were commenced as the air pressure had to be taken off the line and kept off until the cement was thoroughly set.

The plant used consisted of a derrick barge and a barge with an 8-inch belt-driven sand pump. A diver was in constant attendance.

The method of placing the form and filling with grout was as follows: The barges were anchored at the leak and the overlying sand and mud first pumped off the pipe. Then to the flange coupling on the form was connected a 45-foot length of 4-inch pipe. Three rope lines were fastened to the front of the lower jaw of the form, one to the end of the closing chain and one near each side. The diver then took all three of these lines down around under the main and back up on the barge where a man was stationed at each line. Then, as the form was lowered away by the derrick with a line from the 4-inch pipe, the men took in on these lines and the lower half of the form, which dropped open when the form was picked up, was guided into place under the main. By lowering the upper half the form was closed. The piling was then set in the holes provided in the form and were driven flush with the top and the iron straps bolted over them. A jet was then put into the form through the 18-inch square openings and any mud in the form driven out and the joint thoroughly washed off. A piece of raveled, loose rope yarn was then tied securely around the leak to keep the cement

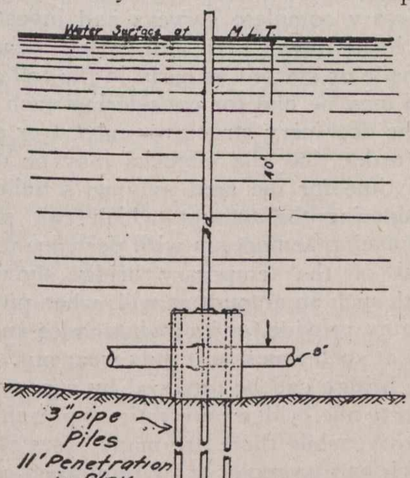

Fig. 3.—Form in Place.

from entering the main. The false forms over the ends of the form were then driven into place around the pipe and the form was ready for cement. The cement was mixed with salt water to a thickness that would just pour through a funnel into the 4-inch pipe leading down into the form. It was found necessary to pour it slowly in order to give it time to settle. Displaced water went out of the form through the holes around the piling in the top of the form. When the form was filled, the nuts on the bolts in the flange union fastened to the form were taken off and the 4-inch filling pipe removed, and the job was finished.

On the last leaks which were closed, pouring the cement through the pipe was abandoned owing to too much lost time in waiting for it to settle. The pipe, however, was still used to lower the form and to hold the form in position until piling were driven. After this it was taken off and the cement, which was mixed as thick as possible, was lowered down to the diver in buckets and poured into the form through the hole at which the pipe had connected.

Where the soft mud and silt was so bad that it could not be kept out of the form, a 4-inch centrifugal pump with a flexible suction end was used to clean out the form after it was in place and all closed, the mud being pumped out through the cement filling hole while a jet alongside stirred it up. In the work of closing these leaks it was found necessary to have the pipe and inside of the form absolutely clean, so that the cement would adhere to the pipe. It was found necessary to take the form off the first leak and do the work all over again, as mud had been pocketed in the grout.

When repairs were completed, all cement was allowed to set a week. About 40 pounds of air pressure was then put on the main and kept on for over an hour. During this time not a single air bubble could be seen and the water meter showed the leaks had been stopped.

Four leaks were repaired and the entire work was executed in about four weeks, including the time of assembling plant, dredging, building forms, etc. By actual time a form was lowered and fastened around the pipe in forty-five minutes. To close the false end gates of the form required fifty minutes. To drive the piles required from one to one and a half hours, depending on how hard the driving was. To mix the cement and fill the form required one hour and fifteen minutes. The total cost of the repair work, closing four leaks, was $2,300.

CARE OF RAILWAY BRIDGES.*

BRIDGE work is a perpetual and continuous job like track and all other classes of railroad work, but the kind and amount of the different varieties of the work changes with the seasons. In the winter surveys should be completed and plans made for future work, and maintenance should be kept up at the least possible expense. Light construction work should be dropped while heavy construction work on abutments, piers, and mass concrete can be pushed to advantage, especially in localities where the ice is strong enough to be of assistance in handling the work, and also where low water is necessary.

In the spring all bridges should be closely inspected and the necessary repairs ordered. Various methods for making inspections are in vogue. The general inspection may be annual or semi-annual, and be added to by periodical local inspections. When semi-annual, the fall inspection is made with a view of seeing that everything is in shape for the winter and to decide on the construction or heavy maintenance work to be considered and investigated for the next season's operations, while the spring inspection is for the purpose of planning and starting the work to be done in the immediate future.

On many roads the bridge engineer makes the general inspection of the large bridges and permanent work, while the inspection of the smaller temporary or wooden structures is left to the local officers. On at least one of the large western roads this process is reversed and the bridge engineer inspects the temporary structures yearly, leaving the permanent structures for the local officers. Possibly it would be well for bridge engineers to combine the two methods and inspect all bridges.

The maintenance and construction forces should be built up and work started as early in the spring as possible so that the beginning of summer will see the work well under way. All pile drivers, machinery and tools should be overhauled and repaired; material ordered, delivered and unloaded, and complete preparations made so that once work is started it can be pushed ahead without delay. A definite programme in the delivery of material to each bridge or each job of work should be outlined and insisted on so that there need be no delays waiting for material; and it should be so arranged that gangs can fully complete each job and then go to the next job without any delay.

*Railway Age Gazette.

The summer is the season for doing the systematic work of repairing, renewing, filling or replacing with permanent structures every bridge on the line as it may need. The work is done by gangs of various sizes which may be either permanent or extra gangs as the work may demand. The pile driver is generally handled by a regular gang, although the method of having each bridge gang educated so that it can also handle the pile driver is sometimes advocated. However, as each road tries to get along with a minimum number of pile drivers so that it is desirable to keep them working at their maximum efficiency at all times, this latter method is of doubtful economy.

On account of the scarcity of foremen and of labor and the advantage of getting work done immediately with as little travelling as possible the combining of the bridge work with section work, signal work and other maintenance work is beginning to be advocated. There are many strong arguments in favor of this and should it be found successful it is likely to reorganize our entire maintenance system and methods. The past 15 years have witnessed very radical changes in the construction of permanent waterway openings due to the use of concrete and steel, and the future will be likely to add to these and thereby also change our methods of maintenance.

As temporary bridge structures are replaced by steel and concrete the amount of maintenance work is very materially decreased. Bridge gangs are replaced with carpenter gangs, painters' gangs, plumbing gangs, etc., as development of the country necessitates. The building up of towns and cities makes it necessary to do much more work around the station grounds than formerly, and the quality and kind of work varies with the nature of the public improvements. Sewers, pavements, permanent platforms, water supplies, plumbing, electric lighting, electric power and other features of latter day progress make it necessary for the railroads to employ specialists who can best handle the necessary work.

Small jobs of construction work and indeed all construction work that it is possible for them to handle should be done by the regular maintenance organization, but large construction jobs require a separate organization which should be flexible as to size and which can be moved from place to place as exigency requires.

Permanent structures in the past have been largely put in by contractors, mainly for the reason that the railroads have not had the necessary equipment for handling the work. However, as they have become larger and permanent work has become more general, it is now becoming customary for the railroads to do their own masonry and steel erection work. Whether they save money in all cases by doing this is questionable, when the cost of the equipment with interest and depreciation is taken into consideration. Undoubtedly where construction work is continued from year to year requiring permanent forces, outfits and machinery, they save the contractor's profit, but in many cases the amount of work and the inexperience of the men makes the cost more to the railroad company, although this fact may not be evident on the surface of their accounts.

During the summer months all the bridge work should be pushed to completion as rapidly as possible, so that in the fall all that remains to do will be to get every bridge and opening in shape for the winter. This not only means that its strength and condition should be cared for, but that the waterways themselves, including the channels and ditches, should be clean and free from obstructions, so that there will be a free flow of water to and from the openings, that the openings may fulfil the purpose for which they were constructed.

As most of the railroads in this country were constructed in a time when timber was cheap, many pile and trestle bridges and timber culverts were built which are being replaced more or less rapidly with permanent structures. This has been accelerated of late years by the use of concrete and the consequent cheapening of the permanent openings. The life of the timber bridges has also been lengthened in many cases by the use of creosoted timber. This material is especially applicable in cases where the bridge decks have to be replaced oftener than the piles, and many years are often added to the life of the bridge by the use of a creosoted deck which may be filled in and ballasted.

When the original timber bridges were built, but little attention was paid to the size of opening required to properly carry the water, so long as it was large enough.

With permanent structures this is not a sufficient rule on account of the greater cost, and the size of openings should be proportioned to the use required of them. This makes necessary complete surveys and investigations of the bridge, its drainage area and outlet. These surveys should be made or started as early in the fall as possible, so that time may be had for considering each bridge and designing the necessary structure, culvert or pipe for the opening in order that the material may be ordered and delivered in time for the next summer's building.

In considering the amount which can profitably be spent for replacing temporary with permanent structures, the first cost of the temporary bridge should be taken together with such an amount as will, when put at interest at current rates, provide for its maintenance and periodical replacement at such times as it may wear out. A common wooden pile bridge can be replaced by a permanent reinforced concrete pile bridge on a ratio at present-day prices of about 3 to 1, while there are many cases of bridges or trestles which can be replaced by reinforced concrete culverts, boxes or pipes for even less than the cost of a wooden structure. The use of reinforced concrete for many railroad structures is growing rapidly and merits the full investigation of every railroad engineer. Pipes, culverts, boxes, highway bridges, subways and overcrossings, arches, trestles, bridges and retaining walls are now being permanently and cheaply constructed of this material, to say nothing of buildings, tanks, coal chutes and other uses for which it is being rapidly adopted.

RAILROADS OF BRITISH COLUMBIA.

It is estimated that during 1913, 650 miles of new railway, not including double-tracking, were laid in British Columbia, of which 285 miles are credited to the Grand Trunk Pacific, 212 to the Canadian Northern, 75 to the Kettle Valley Railway, 65 to the Canadian Pacific, 19 to the Esquimalt and Nanaimo Railway, 18 to the Pacific and Great Eastern, 5 to the Victoria, Vancouver and Eastern, and one-half mile to the Western Canada Power Co. There were also 66 miles of double-tracking, 59 of which is credited to the Canadian Pacific, and 7 to the Victoria, Vancouver and Eastern.

It is announced in the Scientific American that earthquake construction has now reached a very practical stage in the seismic districts of Italy, where all new buildings are being erected under strict supervision with regard to their ability to resist earthquake shocks. Professor Omori, the Japanese authority, has estimated that 99.8 per cent. of the deaths in the great Messina earthquake of 1908 would have been prevented if the buildings had been properly constructed.

A METHOD OF FIGURING REACTIONS FOR CONTINUOUS BEAMS.

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

THE subject of continuous beams has received considerable attention in several of the latest text-books. The methods usually employed, however, are not, in all cases, conducive to their practical application. Only a limited number of cases for different loadings can be found already worked out and for this reason a rigid application of the theory is often avoided in practice. Rapidity, accuracy and ease with which results are obtained, are the requisites of any engineering practice, and it is hoped that the following method will promote all of these, as well as stimulate those who are interested to add more to what is here presented.

The general equation for the theorem of three moments for beams of constant modulus of elasticity and moment of inertia can be written as follows: (See article by Prof. Slocum in the Engineering News, Feb. 19, 1914.)

$$M_1l_1 + 2M_2(l_1 + l_2) + M_3l_2 = -\Sigma P_1 l_1^2 (k_1 - k_1^3) - \Sigma P_2 l_2^2(2k_2 - 3k_2^2 + k_2^3) - 6EI\left(\frac{h_1 - h_2}{l_1} + \frac{h_3 - h_2}{l_2}\right);$$

where E is the modulus of elasticity and I the moment of inertia of the sectional area. The remaining notation may be obtained from Fig. 1.

If the supports are on the same level, and the load P_1 is unity and P_2 is zero, then the formula becomes:

$$M_1l_1 + 2M_2(l_1 + l_2) + M^3l_2 = 1_1^2(k_1 - k_1^3).$$

If P_1 is zero and P_2 is unity, it is,

$$M_1l_1 + 2M_2(l_1 + l_2) + M_3l_2 = l_2^2(2k_2 - 3k_2^2 + k_2^3)$$

In applying these equations to any particular case an influence line may be plotted for the various values of k which lie between 0 and 1. Take the case of a continuous girder with two equal spans. Consider the load of unity in the first span only. Since $M_1 = 0$ and $M_3 = 0$

$$2M_2(l + l) = l_1^2(k_1 - k_1^3)$$

$$M_2 = \frac{l}{4}(k_1 - k_1^3).$$

$$\text{Then, } l R = \frac{l}{4}(k_1 - k_1^3) + l\,(1 - k_1).$$

$$R_1 = 1 - 5/4k + \tfrac{1}{4}k^3 \text{ upwards}$$
$$R_3 = \tfrac{1}{4}(k - k^3) \text{ downwards}$$
$$R_2 = \tfrac{1}{2}(3k - k^3) \text{ upwards.}$$

The following table gives the values of R_1, R_2, and R_3 for values of k from 0 to 1 with the load of unity in the first span only:

k	R_1	R_2	R_3
.0	+1.0000	+ .0000	—.0000
.1	+ .8753	+ .1495	—.0247
.2	+ .7520	+ .2960	—.0480
.3	+ .6318	+ .4365	—.0683
.4	+ .5160	+ .5680	—.0840
.5	+ .4062	+ .6875	—.0937
.6	+ .3040	+ .7920	—.0960
.7	+ .2108	+ .8785	—.0893
.8	+ .1280	+ .9440	—.0720
.9	+ .0572	+ .9855	—.0427
1.0	+ .0000	+1.0000	—.0000

From this table the influence line shown in Fig. 2 may be plotted.

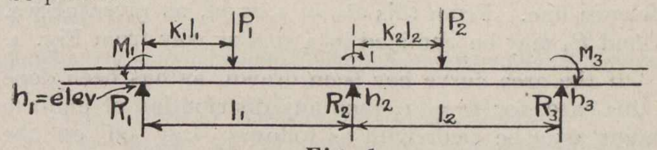

Fig. 1.

In this case it is not necessary to work out a table for the unit load in the second span because the curves are symmetrical, as may be seen in the figure. Since the influence line for any particular number of supports and ratio of spans is always the same, after it is once plotted it may be used to solve problems for any loading.

Problem 1.—Consider a continuous girder with two equal spans, with a uniformly distributed load of w pounds per linear foot over both spans. Since the product of the ordinate to the influence line at any point and the load at that point will give the value of the reaction for which the influence line is drawn, it is only necessary, in this case, to find the area under the curve. The areas below the horizontal axis are considered to be negative areas.

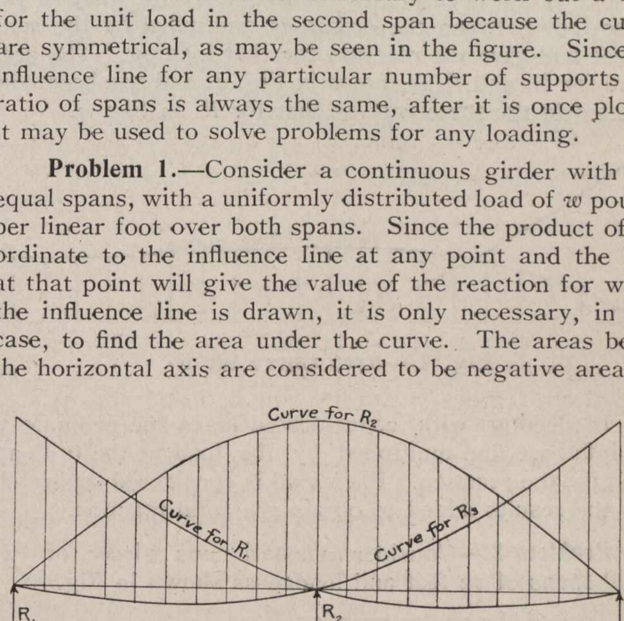

Fig. 2.—Influence Lines for Reactions, Two Equal Spans.

For those who are familiar with the calculus, it is sufficient to say that the area may be quickly obtained by integrating each curve from $k = 0$ to $k = 1$. This will be found to give the well-known values:

$$R_1 = \tfrac{3}{8}wl \text{ upwards,}$$
$$R_2 = 5/4wl \text{ upwards,}$$
$$R_3 = \tfrac{3}{8}wl \text{ upwards.}$$

The following graphical method is found to be more convenient as well as useful. It consists of drawing a curve such that the ordinate at any point will give the area under the influence line between the ordinate and the reaction R_1. This curve, which has been called the area curve for the lack of a better name, is also the same for any particular number of supports and ratio of spans and therefore when once obtained may be applied for any loading. The influence line is plotted first, as shown in Fig. 3, and each ordinate has been multiplied by w. The

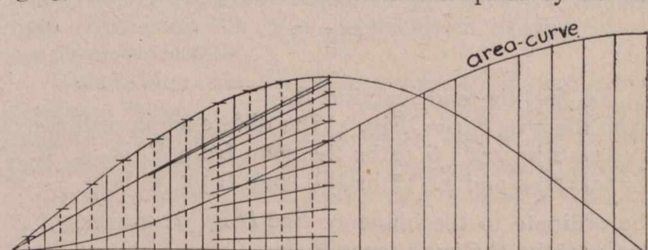

Fig. 3.—Area Curve for R_2.

middle ordinate is then drawn for each strip and horizontal lines are drawn to intersect the vertical ordinate at R_2. R_1 is chosen as a convenient pole and a new curve is constructed, as shown by drawing a string parallel to each ray in turn until it intersects the ordinate which bounds each strip on the left. The last ordinate on the new curve is the total area under the influence line. In this case it is 1.25 w on the same scale as the ordinates to the influence line. From this $R_2 = 1.25\,wl$, as given above. R_1 and R_3 may be obtained in a similar way from Fig. 4.

If the area curve has been drawn, as has been done in this case, for $w = 1$, then any distribution of uniform loading may be dealt with as follows: Lay off on the span the part which the uniform load covers and draw the ordinates at each end to the area curve. The difference in value of these ordinates multiplied by the loading per linear foot between the ordinates multiplied by the span in feet will give the reaction for that loading. The area curve for R_1 and R_3 is shown in Fig. 4, and has been constructed in the same way as that for R_2 in Fig. 3.

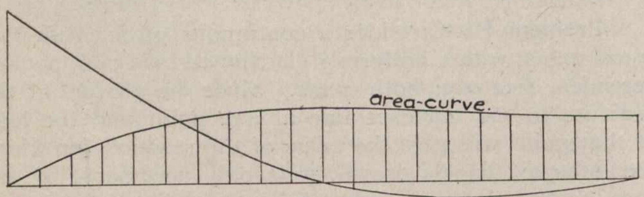

Fig. 4.—Area Curve for R_1.

In dealing with concentrated loads the ordinate to the influence line multiplied by the load at the ordinate will give the reaction. Each load is considered separately and the results added to obtain the total reaction.

Problem 2.—Consider a continuous girder of two equal spans of 12 feet and loaded as shown in Fig. 5.

To find the centre reaction R:

From the area curve (Fig. 3) the difference in ordinates at
$k = 1/12$th and $k = 6/12$ths the span from R_1 is $+.180$
$k = 2/12$ths and $k = 11/12$ths the span from R_2 is $+.460$

The ordinate to the influence line (Fig. 2) at
$k = 9/12$ths the span from R_1 is - - - $+.918$
$k = 8/12$ths the span from R_2 is - - - $+.470$

$+.180 \times 200$ lb./1 $\times 12' = +432$
$+.460 \times 100$ lb./1 $\times 12' = +552$
$+.918 \times 500 = +459$
$+.470 \times 200 = +\ 94$

$R_2 =$ Total $= +1537$ pounds upward for the loading given.

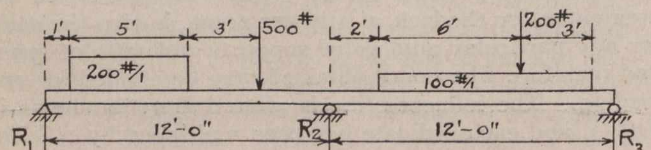

Fig. 5.

To find the end reaction R_1

From the area curve (Fig. 4) the difference in ordinates at
$k = 1/12$th and $6/12$ths the span from R_1 is $+.261$
$k = 2/12$ths and $11/12$ths the span from R_2 is $—.061$

The ordinate to the influence line (Fig. 2) at
$k = 9/12$ths the span from R_1 is - - - $+.166$
$k = 8/12$ths the span from R_2 is - - - $—.078$

$+.265 \times 200$ lb./1 $\times 12 = +636$
$—.061 \times 100$ lb./1 $\times 12 = —\ 73$
$+.166 \times 500 = +\ 83$
$—.078 \times 200 = —\ 16$

$R_1 =$ Total $= +730$ pounds upward for the loading given.

To find the end reaction R_3

In this case the curves in Fig. 4 must be reversed or the loading and reactions may be reversed. Therefore, take R_3 to the left.

From the area curve (Fig. 4) the difference in ordinates at
$k = 1/12$th and $10/12$ths the span from R_3 is - $+.347$
$k = 3/12$ths and $11/12$ths the span from R_2 is - $—.034$

The ordinate to the influence line (Fig. 2) at
$k = 8/12$ths the span from R_2 is - - - $+.595$
$k = .9/12$ths the span from R_1 is - - - $—.058$

$+.347 \times 100$ lb./1 $\times 12 = +417$
$—.034 \times 200$ lb./1 $\times 12 = —\ 82$
$+.595 \times 200 = +119$
$—.058 \times 500 = —\ 29$

$R_3 =$ Total $= +425$ pounds upward for the loading given.

To check the work, the sum of the three reactions must be equal to the sum of the vertical loads.

Sum of vertical loads = 2,600 pounds downward.
Sum of reactions = 2,592 pounds upward.
Error - - = 8 pounds.

From this it will be seen that the error made by using this method, with no more than ordinary care, is well within the limit allowable for practical work. The diagrams used were drawn to a small scale. Greater accuracy may be obtained by using a larger scale, though a very large scale is not to be recommended because the accuracy of the graphical construction decreases rapidly when the scale becomes excessive.

PRODUCTION OF CASINGHEAD GASOLINE IN CALIFORNIA.

Estimates made recently in connection with the development of casinghead gasoline, as the fluid extracted from wet gas is called on the southern Pacific coast of the United States, show that the production will have increased by the fall of this year about 40 per cent. over what it is now, and, it is believed, nearly 100 per cent. in the next 6 months.

The present output of all the plants in California exceeds 20,000 gallons per day. Compressors are operated in all parts of the state, except in the Midway field. It is only recently that the first compressor plant was installed in Coalinga field, on the Turner Oil Company's property; and this has proven to be a very successful one, turning out about 2,000 gallons daily. Other plants throughout the state are yielding from 500 to 2,000 gallons per day.

At present the Union Oil Company in Santa Maria field is making additions to its plant, so as to give it a capacity of 8,000,000 cubic feet of gas per day. This compressor plant will cost nearly $500,000, and, it is said, will be the largest in the world. Further enlargements are being made also in the Santa Maria field on the plant owned by the Rice Ranch Oil Company and on that of the Purity Gasoline Company, so that, when completed, the former will be capable of producing 2,000 gallons a day and the latter 1,500 gallons.

All this is shown in contrast to the fact that two years ago, in the state of California, about 1,000 gallons per day represented the total output of casinghead gasoline.

The product is very high-grade and is mixed with one to one-and-a-half parts of distillate before it is used commercially. Consequently the production secured for commercial purposes may practically be estimated at 50,000 to 60,000 gallons a day, worth at present wholesale prices about $600,000. The value of the casinghead gasoline output, untreated, is lower, being about $300,000.

Mills Brothers, Toronto, now have their office at 215 Ryrie Bldg., not in the Lumsden Bldg., as stated in a recent issue.

Editorial

TRACKAGE REQUIREMENTS OF INDUSTRIAL ENTERPRISES.

One of the chief problems in connection with factory construction is trackage. We are disposed to term it a "chief" problem for the reason that in designing a layout for factory and industrial enterprises it often happens that neither the architect, engineer nor owner gives the matter of trackage much thought. He relies on the supposition that after the land has been acquired and the building erected the tracks can be run into it without trouble. This is a serious oversight, as an important factor in the success of an industry is the quick and economical handling of the raw material to the factory and of the finished product away from it.

As Mr. G. H. Herrold, C.E., of the Civil Engineers' Society of St. Paul, remarked when discussing Mr. King's paper, appearing, in part, in this issue: "A first-class factory poorly erected with reference to trackage, resulting in awkward switching facilities, is only half a factory."

In designing such trackage, it is necessary for the engineer to be familiar with the daily car requirements of the establishment and also with railway switching methods in general and the particular switching services to be obtained at this point. It is sometimes stated that designers of factories have little conception of permissible grades or curvature for practical switching. This often results in requiring additional land for right-of-way, and a track longer than necessary. Sometimes it also requires the building of trestle work or the making of excessive fills in order to bring the track to the factory at the proper level and give a length of track along the factory, which should be level to permit the hand-shifting of cars, if necessary, during the day.

That the tracking problem is not given some attention by our designing engineers of factories is, of course, generally untrue. Nevertheless, it forms a vital part of factory equipment and constitutes requirements to suit the individual case. The simplest track possible is one switch and a stub track running along the factory, which may serve the purpose for small outputs, although a double-end track is more desirable, as it facilitates better handling of the cars. If the nature of the industry is such as to require a different class of cars for the finished product than for the raw material, arrangements should be such that empties and loads can be placed at the factory doors in the morning in the order required and the loads and empties removed from the other end of the track in the afternoon without disturbing any cars not yet unloaded or being loaded. If the business of the factory is sufficient to require two or more tracks, one should run at each side of the factory, one for raw material loads only and the other for loading and outgoing loads only. This is in line with the general tendency of factory design to begin its processes at one side of the factory and deliver the finished product on the other side, eliminating cross-handling as much as possible.

Referring to the remarks of Mr. Herrold, the economical design of factory buildings involves the following problems in approximately the order given:

A study of the processes to be carried on in the factory.

The design or selection of the machinery.

The layout of the machinery and determination of floor space.

The layout of yard room.

The determination of the daily car requirements.

The investigation of switching service in the locality selected for location.

The design of the track layout desired.

The fitting of buildings and tracks to the ground owned or available by actual surveys.

The general revision of all plans to compromise with conditions.

LONG CONCRETE ARCHES.

In the progressive march of science and engineering there occurs, at intervals that possess a semblance of periodicity, epoch-making events whereby the veil of familiarity is lifted to display before the world some superlative achievement, surpassing all others existent in its class, but almost inevitably destined to experience, some day, the event of being itself similarly surpassed. The literature of the engineering profession makes frequent reference to some new enterprise which is, for a brief period of time, "the largest in existence." Bridges, tunnels, dams, ocean liners, sky-scrapers, canals, hydro-electric power developments, etc., have all at one time or another taken advantage of our susceptibility to regard them, unthinkingly, as the last word in their particular line of engineering enterprise. The frequency of such occurrences, however, has come to be a mark of engineering activity throughout the world.

The design and construction of reinforced concrete bridges serve as an illustrative example of this progress. Since the beginning of the 20th century there have been a dozen or more occasions to refer to some new and surpassing construction throughout the various countries.

In *The Canadian Engineer* for June 27th, 1912, an article was published describing the Ponte del Risorgimento, which, with its arch span of 328 ft. in a 33-ft. rise has for several years maintained its distinction as being the largest reinforced concrete arch in the world. It was built in 1911 over the Tiber in Rome, and its design was noteworthy in that at its crown the thickness of the arch is only 8 in., increasing to 20 in. at the springing lines. Stiffening ribs, 8 in. in thickness, extend throughout its entire length.

This bridge over the Tiber has resigned its distinctive greatness to an arch which is now being constructed at Langwiez, Switzerland, and which has a 330-ft. span. This bridge, however, unlike its predecessor in design, is of massive construction, with a rise of nearly one-half of its length.

The Ponte del Risorgimento, noted above, had taken the title from the Grafton bridge in Auckland, New Zealand. This was a reinforced concrete structure with a 320-ft. span and a rise of 86 ft. begun in 1907 and completed in 1910. The Munro Street bridge in Spokane, Wash., also of concrete, had a span of 285 ft. and a rise of 117 ft. For a list of long concrete arches giving dates of erection, principle dimensions, etc., the reader is referred to *The Canadian Engineer*, Vol. 22, No. 26, page 843.

ECONOMICAL DESIGN OF INDUSTRIAL WORKS.

THE economical design of factories involves the solution of a large number of simple engineering problems and requires a broader knowledge of engineering than most types of buildings. The designer should be an engineer. He should have a fair working knowledge of architecture as well as of civil, structural, mechanical and electrical engineering, and he should acquire a clear understanding of the particular machinery to be installed and the processes to be carried on in the construction he is setting about to design. It is the work of an industrial engineer rather than an architect, but the former should not minimize the importance of the architectural requirements of the work and devote all his attention to the considerations of economy of construction and efficiency of operation. Although the æsthetic considerations are not as important, they should be given due consideration.

In a paper presented to the Civil Engineers' Society of St. Paul by W. E. King, C.E., and published in the June issue of the Association of Engineering Societies, the work of the industrial engineer in the design of industrial structures is discussed and the problems relating thereto are outlined in order to give a general view of the economics of this form of construction. The following is extracted from Mr. King's paper:—

The first consideration is the proper locality for the industry. This is largely an economic problem, which the engineer may or may not be required to study.

The cost of any manufactured commodity to the retailer consists of the following items: Cost of raw materials, cost of the transportation of these raw materials to the factory, cost of labor on materials, cost of power, overhead charges, including interest on money invested, depreciation of plant, insurance, office time and advertising, cost of distribution, profit to the manufacturer.

Assuming the price to be received for any finished commodity to be fixed by competition, then that project which will pay the largest profits, is, of course, the one where the sum of the first six charges is a minimum. This does not necessarily mean that any one item should be reduced to a minimum, but that the sum of all the items taken together is the least possible. The usual difficulty is that some one man almost always plans each project with the idea of reducing some one item to a minimum. For instance, a man who has spent the larger part of his time in handling of workmen will insist that the plant be so located that there will be an abundance of cheap labor. If he had at one time been a purchasing agent he would plan his plant to save all freight possible on raw materials. The sales manager is interested in the location of the factory with respect to the market. The man who furnishes the money is sometimes unduly interested in cutting the first cost down to a minimum, without regard to whether the interest on his money might be larger if more money were invested.

It should be the duty of the engineer to study these questions and to so present them that they will occupy their proper rank of importance. This rank is, of course, different in different kinds of factories. In the fabrication of structural steel, for instance, perhaps the most important factor is freight. This includes freight from the rolling mill to the factory and freight on the finished product from the factory to the consumer. In some parts this freight amounts to more than half the cost of the finished product.

Where freight is one of the chief considerations, charts may be prepared showing the zone in which a product may be profitably marketed. The boundary of each zone will be determined by considering the sum of the freights on raw and finished materials for the proposed location as compared with other possible locations. The properly prepared chart will show the overlapping territories where competing factories sell on an equal basis. It will show the area where the factory in question has the advantage, and it will also indicate the areas which cannot profitably be reached.

The matter of available market probably reaches its greatest importance when the capacity of a profitable factory is about to be increased. It may be that the selling organization is now reaching all of the profitable market zone, and that to increase the sales the product must be marketed at a disadvantage.

Some industries use large amounts of fuel or power, which requirement is the determining factor in their location. In connection with this there is a tendency to group factories around water-power sites, which will probably not be as marked in the future, because our modern methods of electrical transmission allow power to be economically delivered at a considerable distance.

Having determined on the vicinity where the factory is to be built, the next consideration is the purchase of the exact site necessary for the project. The exact area of land which is necessary is usually a troublesome one. Most successful projects are hampered by lack of room to provide for their growing needs. On the other hand, it is a serious handicap to a young industry, to be burdened with heavy interest charges and taxes on land not at the time in use.

The size and shape of the area necessary for present needs is usually determined by making a preliminary plan of the whole project. If the engineer be unfamiliar with the need of the industry in question, this will usually involve quite an extended study of the methods of manufacture used by the particular organization and of similar organizations in other places. If, however, the designer has already prepared plans for other similar plants, the tentative preliminary plans involve only a study of the peculiar requirements of the special case. This preliminary plan should, of course, take in the reasonable growth of the industry, which usually may be approximately obtained by a comparison with similar industries in other communities. With the approximate area required clearly in mind, a search of the locality will usually show a number of available sites. For projects of some importance, plats are usually prepared showing how the proposed sites may be developed. These plats should show the approximate grades and elevations of all adjacent streets, the location of sewer, gas, water and power connections and the available connection to adjacent railroads or sidetracks. If grading of streets or of the lots will be necessary, this should be estimated and added to the comparative price of the lots. For the purposes of comparison the cost of the sidetrack, including necessary grading, the cost of sewer, water, gas and power connections should be obtained. Very often the owners will buy a lot first, without considering the cost of these things, which must be added to make the lot available, and in so doing they fail to get the most economical site. Plats showing the proposed sidetrack should be submitted to the railways interested and assurance should be had from their engineering and contracting departments that they are willing to put in the desired connection. If the sidetrack must cross the public road it is just as well to be sure of the permit before putting money into the lot.

Sidetracks.—At this time a study should be made to determine the number and length of sidetracks which will be required. In general, the track should be long enough to hold as many forty-foot cars as the company will need to load and unload in any one day. In isolated places, where the cars are not set as often as that, the sidetrack must be long enough to allow for all the unloading and loading which must be done between each setting by the switch engine. Sidetracks for loading and unloading should in general be level. The rules of the railroad in question, of the state and interstate railway commissions and the state labor laws are the determining factors in the amount of room required for sidetracks.

Building Plans.—After the exact location of the site is determined, then the plans of the buildings may be prepared. It is a mistake to make the final plans of any building before its definite location is settled. The natural grades of the land itself, the streets, the points of the compass and the condition of the subsoil almost invariably change the plans to such an extent that they must be revised or redrawn. All of these conditions should be determined by an exact survey before work on the plans commences.

As before stated, the basis for the design of the factory building should be a complete understanding of the processes to be carried on in the building. Too many factories are built first and the machinery just put in, one piece at a time, after the building is completed. This usually results in the uneconomical use of the floor space, unused spaces occur in some parts and a congested condition results in other parts.

The first plans to be prepared should be complete machinery plans. A study should be made of the progress of the materials through the shop. In general, the manufacturing processes should be so arranged that there will be no lost motion. The various materials which go to make the finished product should all travel through the various parts of the factory in such a way that they will arrive at the assembling-room without having travelled any greater distance and without having been transferred more times than is absolutely necessary. Leaving the assembling-room, the materials should go by the shortest possible route to the storage and shipping-rooms. This part of the work is, of course, best planned with the prospective superintendent of the shop. It is sometimes difficult to get the benefit of this man's detailed knowledge and experience without letting his narrowness of viewpoint blind the designer to the broader phase of the question.

As a rule, a good factory superintendent has spent the larger part of his life in some one factory. He probably has made that factory a success. That leads him to think that he knows all there is to know about that business. At least he thinks he knows more than any engineer whom the owners can hire. That is generally true, but his difficulty is that he is so close to his job that his perspective is warped. For instance, if ten years ago he tried a belt-conveyor in his factory which he bought and installed improperly himself, and then afterward abandoned because it did not do the work required, he is convinced that he does not want a belt conveyor in his new factory. The fact that belt-conveyors have been improved since he tried them and that there are thousands of them working satisfactorily under similar conditions will impress him only if you can overcome his prejudices. If you can make him feel that he and the engineer are working together to get the best possible design and that you realize the value of his suggestions, then, generally, it is possible to get him to listen to yours.

The building should be built to suit the machinery. The columns and beams, the height of stories, the location of heating and plumbing pipes, the sprinkler system, the natural and artificial lighting should all be arranged to suit the machinery.

The economical arrangement of the structural parts of the building should also be taken into consideration in the arrangement of the machinery. If possible, the columns should not be spaced to suit special machines unless there is some very decided advantage in doing so. It must be remembered that the life of a building is several times the life of the machinery installed, and that the machinery of the future may be entirely different.

Types of Factory Buildings.—There are two types of factory buildings which are here considered separately. The first is the ordinary one-story building with a hip-roof, which may or may not be surmounted by a monitor. It usually has large, unobstructed floor space to provide for the movement of cranes and other large machinery. The second type is the warehouse type of one or more stories in height. Industries which require a clear floor space of more than twenty-five feet in either direction are usually housed in one-story buildings, because it is expensive to carry the weight of upper floors on long spans. Where the materials manufactured are of such size that columns spaced from sixteen feet to twenty-five feet on centres are not objectionable the building of several stories is usually more economical.

A one-story building costs the most per sq. ft. of floor area. This cost per sq. ft. of floor area decreases somewhat with the number of floors built, up to four stories. Above that height the cost per sq. ft. gradually increases. There is comparatively little difference in the cost per sq. ft. of the floor area between a three and an eight-story building.

If basement floor space is suitable it is the cheapest which can be obtained, except where the loads to be carried on the first floor are extremely heavy. A one-story shop building in fireproof construction will cost from from $1.25 to $2.00 per ft. of floor space area, depending upon the height of the story, depth of footings, length of spans and kind of exterior finish used. Fireproof buildings of more than one story may be built for as little as fifty cents per square foot of floor area. These approximate figures do not contemplate any sort of plaster or interior finish except whitewash. They do include a properly finished cement floor. The cost per sq. ft., of course, decreases as the size of the ground plan increases. It is more for a long, narrow building than for a square building. However, a factory building should not be made too wide on account of the difficulty in properly lighting the interior. For ordinary factory work from 40 to 50 ft. is the best width. A building of this width can be lighted with a story height of from 12 to 14 ft. If the width of the building be made from 75 to 100 ft., then the story height should be increased to from 14 to 16 ft., the windows being placed as high as possible.

One-story shop buildings are usually built of what may be termed semi-fireproof construction. They are usually built of materials which will not burn, but cannot be said to be entirely fireproof, because the steel trusses are usually left unprotected, so that they might be damaged in case of fire occurring in the contents of the building. As before stated, the one-story plan is usually adopted where large, unobstructed floor spaces are required. This results in long-span steel trusses supporting the roof.

The most common type of roof is the "A"-shaped roof. This roof has many advantages. It is easy to keep watertight, it clears itself of snow easily, and with monitors or ventilators at the peak provides good ventilation for the factory. If these monitors are made wide enough and are provided with windows they admit considerable light, but if the building is high and wide, monitor windows usually do not admit a satisfactory light.

A better type of roof, where light is essential, is the sawtooth roof. This roof is made up of a series of pitched roofs, rising towards the north and stepping down with a vertical step, in which windows are installed. These windows, facing towards the north, admit a diffused light, which illuminates the floor below without casting shadows. If the windows in the sawtooth construction are arranged to swing, they provide as good ventilation as the old monitor type. The disadvantage with sawtooth construction is that it presents a number of valleys where snow may lodge. In some cases steam pipes have been installed to melt the snow. This serves the purpose, but is rather expensive. In buildings where there is considerable steam in the air condensation gutters are necessary under monitor and sawtooth windows.

Roof.—The most unsatisfactory problem in shop building is probably the roof. It first must be watertight; second, if the building is to be heated in winter, it must be of such material that condensation will not occur on the under side; third, it should be fireproof; fourth, it must compete with a large number of cheap roofs which are lacking in one or all of these qualifications. A standard roof construction consists of three-inch hollow book tile, laid on steel tees. This tile is covered with some good prepared roofing, which is cemented and tacked to the tile. This roof is very expensive, but it fulfills all the requirements stated above. It costs, including supports, about thirty cents per square foot.

Another good roof is 2-inch dressed and matched sheathing, laid on wood or steel purlins and covered with a good prepared roofing. It is just as good and much cheaper than a book tile roof, but, of course, is not fireproof. It will cost about 20 cents per sq. ft., including supports.

In some instances a thin concrete slab laid on steel or concrete purlins has been used. Considerable condensation occurs under such a roof in cold weather. Furthermore, it is very difficult to keep a tin roof slab from being damaged by frost while being laid in cold weather.

If the shop is not to be heated in winter time, corrugated iron laid on steel purlins makes a very inexpensive fireproof roof, costing about 12 cents per square foot in place, including supports. It is fairly watertight, but, of course, is very cold in winter.

There are, of course, many other kinds of roofs, but the price for any roof comes between the limits here given.

There is not so much choice in the materials of construction of the side walls of a building as the roof. They may be of brick, stone, concrete, corrugated iron or glass. In this vicinity brick is the most usual and satisfactory material. Buildings with high stories are usually made with steel frames, the walls being simply curtain walls bricked in between the columns. Hollow brick should be used for the inside layer of brick to prevent condensation of the side walls.

Concrete for side walls is more expensive and less satisfactory than brick. Concrete blocks are sometimes used, and are probably all right where enough cement is put in the blocks. Such walls are, however, weak, due to the lack of bonding between the blocks.

A 12-in. common brick wall in this part of the country will cost about 38 cents per sq. ft. in place. With a good facing brick and some architectural decoration the cost may be increased to from 40 to 60 cents per sq. ft.

In the modern factory building the question of material of the outside walls is not an important feature, because from 75 to 100 per cent. of the wall area is occupied with windows and doors. The old style shop building did not, as a rule, admit enough light. Our new buildings probably admit too much. It is a mistake to assume that a workman needs as much light to work by as there is out under the open sky. Too much light is almost as bad for the eyes as too little. Most of the inconvenience of working indoors comes from working with a strong light from one side which casts shadows. Windows should be so arranged that light will reach every point from at least two directions and be of as near the same intensity in both directions as possible.

Another question upon which there is usually some argument is the kind of windows to be used. The three types most in use are the standard wooden sash, the rolled steel sash and the fire underwriters' sash of sheet steel or copper. The underwriters' sash is very little used for shop buildings because of the expense. They will, however, greatly reduce the insurance rate upon such walls of the building as have a bad fire exposure.

The most satisfactory sash at the present time, for factory work, is the rolled steel sash. This is a comparatively new product, having been on the market for only a few years. Where large areas are to be glazed the small size of the steel muntins and mullions permits the maximum amount of light to enter the building. Several factories have been built with the side walls almost entirely of glass, the only obstructions in the walls being the columns and the brick work at the floor line. Steel sash have a few disadvantages which should be taken into consideration. The ventilation is usually secured by pivoting a part of the sash near the middle. In factories where screens are necessary it is not possible to have ventilation because the screen will not permit the ventilator to swing. In this northern climate storm sash are desirable because of the loss of heat through the glass by conduction. Steel sash are too heavy and too expensive to use for storm sash. If wooden sash be used the advantages obtained by the use of steel inside sash are lost.

The cheapest sash to use is undoubtedly the double hung wooden sash with which we are all familiar.

Four General Classes.—Factory buildings of more than one story naturally divide themselves into four general classes, according to the materials of which they are constructed. This classification is really made by the fire underwriters inasmuch as the different types take different insurance rates. In fact, the rate of insurance is the consideration which most often determines the type of construction.

These classes are, frame construction, slow burning timber construction, structural steel and reinforced concrete.

In the frame construction class should be included all buildings having either brick or timber walls, wherein the floors are of wood and the joists narrow and spaced close together. Such buildings are, of course, the cheapest which can be built. By far the larger number of the present factory buildings are of this type. When an industry is in an experimental stage and the process of manufacture and the machinery are likely to be changed with experience, it is more economical to build in this manner. If a building is anything more than a temporary structure the extreme fire hazard, the danger to employees upon the upper floors and the lack of rigidity for supporting machinery are disadvantages which should be taken into consideration.

In the slow-burning mill building construction, as described by the fire underwriters, the walls must be of brick or stone. It differs from the frame construction in that the joists are spaced from 3 to 6 ft. apart and are timbers of considerable size. The floors are matched planking. All stair and elevator hatchways must be enclosed, with doors at each floor opening. This construction is, of course, somewhat more expensive than the frame construction. Its principal advantage is that it takes an insurance rate about 20% less than frame construction.

Any timber construction has several advantages over more permanent types such as concrete or steel. Alterations in the buildings, due to changes in processes of manufacture and the installation of new machinery, are much more cheaply and rapidly made. The expense of attaching shafting and machinery to the finished structure is considerably less. Wooden buildings are more rapidly constructed than either reinforced concrete or structural steel buildings.

The columns in buildings with wood beams should be spaced from 12 to 18 ft. on centres. If a greater column spacing than this is required it is usually more economical to make the beams spanning in the longer direction of steel. These beams may rest on cast iron or steel columns, the remainder of the construction being of wood.

A better construction consists of steel columns and beams throughout. The floors may then be made of reinforced concrete or tile. If the columns and beams are then covered with fireproof material such as tile or concrete the building may be regarded as the best type of building which modern civilization has produced. In such a building the steel columns do not occupy so large a percentage of the floor area as do concrete columns. Exact stresses in a steel frame building are more easily computed. The chance for variation in the strength of the material due to faulty workmanship or design is not nearly so great. Alterations of the building are more economically made in a steel than in a concrete building.

The most popular type of factory building in many localities is reinforced concrete. A properly designed concrete building is the very best building which can be put up for many industries. It is entirely fireproof and takes the same rate of insurance as a fireproof steel building. Such buildings are probably the most rigid type which can be constructed. The material will stand a large amount of abuse in the way of faulty workmanship and design. Other types of buildings deteriorate with age, but a concrete building increases in strength. So far as we are able to determine at this time, our concrete building will be as good in the structural parts fifty years from to-day as they were when built.

The floor spans of a concrete building may economically be made from 16 ft. to 24 ft. in length. The exact span for minimum cost, of course, depends upon the expense of the foundations. The more expensive the foundation piers the longer may be the economical span. We find that the flat plate type of column pier is considerably less expensive than the old-fashioned masonry piers of the pyramid type.

The statements made concerning the exterior walls of one-story buildings are, of course, true in regard to buildings with a greater number of stories. It is usually economical to build self-supporting exterior walls for buildings up to three stories in height. For buildings higher than three stories the walls are often made twelve inches thick and carried upon the steel or concrete frame of the building.

The details of the work which have been described in the foregoing paragraphs are interesting in illustrating the method by which we try to arrive at the final economical design. A factory is made up of such a large number of details that only a few can be touched upon at this time. The arrangement of electric lighting and the ventilating systems, so as to give each worker sufficient air and light, are other interesting problems.

Finally, it may be said that in the last analysis the most economical factory building is the one where each worker is given the best conditions for doing his work, for the least cost.

UNION STATION, TORONTO.

The contract for the construction of the new Union Station for the city of Toronto was awarded last week to the P. Lyall Construction Company, of Montreal and Toronto, the amount of the contract being approximately $4,000,000. The station is being built by the Toronto Terminals Railway Company, Mr. J. R. W. Ambrose, chief engineer. Messrs. J. M. R. Fairbairn, assistant chief engineer, C.P.R., and H. R. Safford, chief engineer, G.T.R., are consulting engineers to the organization formed for the purpose of handling the enterprise.

The new station will be constructed on a site east of the present building and will be bounded on the north, east and west by Front, Bay and York Streets respectively. The site forms a portion of the fire-swept region which has remained for the most part unoccupied and desolate since the conflagration of 1904.

Messrs. Ross, Macdonald and Jones are the architects with whom is associated Mr. J. M. Lyle, Toronto. The erection of the new Union Station will form a part of a $15,000,000 development project which includes a large amount of grade separation to be affected along the water front.

REINFORCED CONCRETE SEWER AT VICTORIA, B.C.

A reinforced concrete sewer is under construction in Victoria, B.C., that will drain a section of the city 425 acres in extent; also, 800 acres in Saanich and 1,000 acres in Esquimalt. It will be two miles in length when completed, and will empty into an outfall near Macaulay Point, where it will discharge through several hundred feet of steel pipe at a point below water level where the tides admit of unusually favorable disposal. The trunk sewer is 27 in. in diameter at its beginning, increasing to 36 in. at the outfall, and the sections of reinforced concrete pipe are cast in 5-ft. lengths.

The excavation for the sewer consisted of about 7,250 ft. of rock tunnelling and 3,100 ft. of open work. The tunnel work attains a depth of 65 ft. below the surface, and consists essentially of three separate tunnels, the first of which is now being driven and in connection with which two shafts have been sunk. The tunnel is being constructed 5 ft. in width and 7 ft. in height. The excavated material is for the most part solid rock, and admits of an average progress of 5 ft. per day. The work is in charge of Mr. A. E. Forman, assistant to City Engineer Rust.

Another notable reinforced concrete pipe line in Victoria is that in connection with its water supply. It is a 28-mile conduit, 42 in. in diameter, and is being constructed by the Pacific Lock-Joint Pipe Co., of Tacoma.

Coast to Coast

Prince Rupert, B.C.—The G.T.P. Railway officials expect to have the big dry dock at Prince Rupert completed before the end of the present year.

Edmonton, Alta.—Work on 137,000 square yards of paving and concrete sidewalks will commence almost immediately. The work will cost in the neighborhood of $400,000.

Windsor, Ont.—As a result of the decision handed down in the High Court in Toronto, all work on the city's new incinerator plant at Windsor has been abandoned temporarily.

Brantford, Ont.—It is reported that by August 1 negotiations will have been completed, and both the Brantford Street Railway and the Grand Valley Railway will belong to the city of Brantford.

Moose Jaw, Alta.—A recent report states that the C.P.R. line of 70 miles from Kerrobert to Monitor, Alta., will be completed shortly. This will establish through connection from Moose Jaw to Lacombe, Alta.

Halifax, N.S.—Telephonic communication with Prince Edward Island on a commercial basis was formally inaugurated on July 7. The new cable was laid by the Maritime Telegraph and Telephone Company.

East York Township, York County, Ont.—It was announced at the meeting of East York township ratepayers that the construction work on the hydro-electric power line extension up the Don road would be commenced immediately.

MacLeod, Alta.—It is expected that the Calgary MacLeod branch line of the C.N.R. will be completed this season. The new line will run directly north to Red Deer, continuing north several miles west of Lacombe and Edmonton to Athabasca Landing.

Brandon, Man.—No definite decision was arrived at by the Railway Commission at its recent sitting in Brandon in reference to the city's application for an order directing the Grand Trunk Pacific Railway Company and the Canadian Northern Railway Company to establish joint terminals in Brandon.

Saskatoon, Sask.—A great many gas and oil leases have been filed during the past few weeks at the Dominion lands office in Saskatoon. The district in which the lands leased are located is an area of over 60,000 acres, partly in the Hanley district and partly in the townships on the boundary or near the Hanley district.

Victoria, B.C.—Arrangements have been completed by the contractors, Messrs. Grant, Smith, and McDonnell, for the immediate shipment of rock to the piers under construction by that firm at Ogden Point from the new quarry which has opened up at Esquimalt. More rapid progress will now be made upon the work.

Hamilton, Ont.—The works committee of the Hamilton council has decided to recommend to the board of control that a by-law be submitted next January to ratepayers regarding the purchase of a stone quarry, as well as with reference to the offer of the Canada Crushed Stone Corporation to supply stone to the city at from 85 to 90 cents per ton.

Winnipeg, Man.—A communication from the Dominion public works department to the Winnipeg and St. Boniface harbor commission, stated that parliament has granted the sum of $200,000 for harbor improvements to be effected on the Red River at Winnipeg and St. Boniface. Work on the docks is to be commenced and carried on as rapidly as possible.

Victoria, B.C.—According to the assertion of H. E. Beasley, general superintendent of the E. and N. Railroad, the opening of the line for service will take place between July 25 and August 1. The construction of the Trent River bridge has been completed, and the laying of rails and ballasting is practically completed into Courtenay, the temporary northern terminal of the line.

Weyburn, Sask.—Within the coming year, the town of Weyburn expects to acquire connection with three railways. The C.N.R. has promised construction of a link to that centre. Through trains from Winnipeg to St. Paul to be operated over the C.P.R. cut-off to Vancouver are now definitely promised. And the G.T.P. is already completing plans for opening up the territory to the southeast of Weyburn, and the position of the town as a distributing centre is expected to be assured.

Montreal, Que.—The G.T.R. Company has completed the installation on the Victoria Jubilee Bridge across the St. Lawrence River, a new type of automatic train signals, which is the first of the kind to be used in Canada. The new apparatus, known as the alternating current, 3-position semaphore, consists of eleven signals, the total length of the first line covered being 3½ miles. It replaces the Hall banjo type, which was installed in 1902 and which was at that time the first automatic signal put in use in Canada.

Vancouver, B.C.—The plans for the Dominion Government drill hall, which is to be erected at Vancouver at a cost of $350,000, have been completed by Perry and Fowler of Vancouver, and are now on inspection. The building measures 278 by 174 feet. The drill hall proper, measuring 225 feet by 120, will extend up to the roof of the building with a sweeping arch above. It will be well lighted and ventilated. Surrounding the drill hall on the main floor will be company armories and officers' rooms. The basement will provide a swimming pool, gymnasium, miniature rifle ranges and bowling alleys as well as ordinance store rooms, armory repair shop and sheds of different descriptions.

Chatham, Ont.—Chatham is experiencing considerable difficulty in arriving at a solution to its power problem. The Chatham Gas and Electric Company has refused to accept the proposition recommended a short time ago by the Hydro-Electric Commission that the city purchase the plant of the gas company for the sum of $410,000. The last offer of the company to the city is to reduce the rates of electricity to a basis equal to the probable rate that Hydro power would be sold at in Chatham, run the plant for 10 years, and then take the accrued profits and turn the plant over to the city without the expenditure of a dollar. The city council is now considering the advisability of accepting this proposition.

Montreal, Que.—At the Maisonneuve plant of the Canadian Vickers Company will commence on August 1 the construction of what will be the second largest icebreaker in the world, and will cost $1,000,000. It is to be used by the Canadian Government for icebreaking in the river and gulf of St. Lawrence, and is to be launched next May. It will be of the heaviest type of construction, will carry a crew of 90 men and will have the following dimensions: length, 292 feet; width, 56 feet; draft, 19 feet. It will be equipped with 8,000 horsepower engines. To form the launching ways between 800,000 and 900,000 feet of southern pitch pine is being brought to Montreal from Georgia in a vessel which sailed for Montreal on July 4th.

St. Hyacinthe, Que.—Within 35 miles of Montreal, across the St. Lawrence River, and close to the town of St. Hyacinthe, drillers are boring for natural gas under the direction of the promoters, the Canadian Natural Gas Company. The company has secured the right to operate throughout the counties of St. Hyacinthe and Richelieu, an area of 250 square miles. One well was sunk to a distance of 1,860 feet, and produced a flame when lighted of from 75 to 100 feet in height, giving a pressure from a 2½-inch pipe, of approximately 600 pounds to the square inch. Encouraged by this success, the company extended its operations and have now almost completed the sinking of another well. The power for operating the machinery for this second well comes from the gas flowing from the first. It is the company's intention to sink a third well this summer, so as to ascertain definitely the extent of the "find."

Squamish, B.C.—The P.G.E. Railway Company has appointed Mr. J. Cumming in full charge of all the harbor and railway development work to be carried out by the company at Squamish. The harbor work will involve changes in the channel at the mouth of the Squamish River at its entrance to the Sound. The plan is to alter the course of various channels in such manner as to bring the river up against the base of the mountain and thus reduce erosion. The entire plans for this big undertaking will not be finished for several months, but in the meantime it will be possible to proceed with filling and banking work, and the clearing of the Indian reserves recently acquired by the company at Squamish. The reserves contain altogether 1,100 acres, from which should be deducted 200 acres of steep mountain sides. Besides these 900 acres, however, the company is filling in 200 acres of tideflats on the southwest waterfront of the old townsite of Newport. Wharves and waterfronts are to be laid out by Mr. Cumming, as well as a wide boulevard on one side of the old townsite.

Winnipeg, Man.—It has been announced that the administration board of the Greater Winnipeg Water District will call very shortly for tenders in connection with the various works in connection with the 84.72 miles of aqueduct for the Shoal Lake water project, which will cost approximately in totum $8,729,000. Advertisements are to be placed in journals in Canada, the United States, England, France, and Germany. Last year a division of cost of the work was made by the consulting engineers of New York and Boston as follows:—1,880,000 cu. yds. earth excavation west of the Summit Cut, at 60c., $1,116,000; 1,100,000 cu. yds. earth excavation in Summit Cut, at 75c., $825,000; 94,000 cu. yds. of rock excavation, at $2.50, $235,000; 2,300,000 cu. yds. refilling and embankment at 40c., $920,000; 340,000 cu. yds. concrete at $13.00, $4,420,000; 29,000 cu. yds. reinforced concrete at $17.00, $493,000; 13,000 ft. timber platform at $40, $520,000; steel for reinforcing aqueduct, $70,000; special work at and near river crossings, including waste weirs, $80,000; gate and screen chamber and other works at intake, $50,000; total for concrete aqueduct and appurtenances, $8,729,000.

Victoria, B.C.—An announcement has been made by J. S. Maclachlan, Government resident engineer, to the effect that by the end of July, the first signs of the Ogden Point breakwater, being constructed by Sir John Jackson, Limited, will appear above water. Previously, difficulty has been experienced in the laying of the granite boulders which form the foundation, owing to the inclined surface of the sea bottom at the inner end of the breakwater. But the work has advanced, nevertheless, satisfactorily; and, according to the engineers' statistics, the weight of granite blocks laid since the operations were started amounts to 3,677 tons. Throughout the month of June the divers placed in position a total of 1,199 tons. June has also been a record month in the dumping of rubble on the breakwater site, 60,606 tons having been dumped. This exceeds that of the previous month by 10,000 tons. The total amount of rubble now dumped is placed at 373,608 tons, which now comes to within 20 feet of low water as far as the final stretch, or the last 700 feet of the breakwater. An idea of the work which has been done may be gleaned from a consideration of the base of the breakwater, which is 200 feet in width and tapers up to a height of 72 feet. The contract is so far advanced that actual operations will be started this month on the concrete work forming the superstructure of the great sea wall. At the present time 1,250 barrels of cement have been delivered on the site by the Associated Cement Company, of Bamberton, which has been awarded the contract to supply all the cement necessary for the breakwater. Before being used this cement is to be given a 28-day test. The leveling off of the wharf area is now practically complete, 1,717 cubic yards having been leveled off last month. And the fifth and last of the dolphins has been driven, marking the extreme end of the sea wall.

Vernon, B.C.—A publication from Vernon states that July will see the commencement of construction on the branch line of the C.N.R., running from Kamloops to the Okanagan Valley. The Hon. Price Ellison, provincial minister of finance, has signed the $5,110,000 guarantee passed by the British Columbia Legislature in February; and the Hon. Mr. White, Dominion minister of finance, has signed the guarantee of the Dominion Government to the C.N.R. amounting to $45,000,000. The $50,000,000 worth of bonds which have thus been made available are to be marketed at once in England. Operations will be begun simultaneously at three points, Vernon, Armstrong and Kamloops. From Vernon construction work will proceed in four directions; from Vernon toward Armstrong, toward Kelowna, toward Okanagan Landing, and up the White Valley toward Lumby and Shuswap Falls. The entire branch line will be about 148 miles in length. From Kamloops to Vernon the survey is 81 miles long, from Vernon to Kelowna 35 miles, from Vernon to Okanagan Landing about 4 miles, from Vernon to Lumby 17 miles and from Lumby to Shuswap Falls 10 miles. The last named 10-mile extension is not included under the recent guarantee, but will be built to reach the company's power site and townsite at the falls, where electric power is to be developed to operate the Lumby and Kelowna lines and probably the entire line through to Kamloops. Active development work on the power site will probably not be begun until next spring. But in every way possible, the work will be hastened; and grading, with the exception of the few heavy pieces of rock work on the Kamloops-Okanagan line, is expected to be practically completed by January 1, 1915. In connection with the section from Vernon to Okanagan Landing, the C.N.R. plans to build a lake fleet of passenger steamers and freight barges, which will increase the traffic from this district.

PROGRESS OF CONSTRUCTION ON THE P.G.E. RAILWAY.

A report upon the work done up to date upon the P.G.E. Railway, was made recently by A. H. Sperry, general manager of the railway company, to D'Arcy Tate, vice-president of the company, and to the Premier of British Columbia, Sir Richard McBride. Mr. P. F. Welch, of the firm of Foley, Welch and Stewart, the contractors for the railway construction, has chief charge of the work, and has in his employ 5,600 men. The work on the road is being rushed as fast as possible to an early completion. It is planned to have the grade work completed all the way to Fort George this year, before the frost enters the ground. It is stated that an excellent standard of efficiency is being maintained in every detail of the work along the entire 810 miles of length of the railway from Squamish to Peace River, via Fort George. Mr. Sperry reported that for a distance of 13 miles from Vancouver, grading and tracklaying have been completed and that on July 1 next, a local service will be inaugurated in that section of the country.

There is a gap of some 20 miles between Horseshoe Bay and Squamish, where heavy rock work will be necessary. This section will be left by the construction gangs until all the other portions of the contract between Fort George and Vancouver are well on towards completion. It will then be undertaken and promptly finished.

Between Squamish and Clinton, where the heaviest work has been encountered, the grade is all but completed. Steel is laid as far as Cheakamus River; and as soon as the bridge is completed at this point, steel will be laid right through to the Pemberton Meadows country, and thence through to Lillooet.

It is intended that ballasting shall go on simultaneously with operations of construction, so that the road will thus be put into operation with the minimum of delay. There has been considerable delay in the section of Cheakamus, but this part of the line necessitates the construction of 12 bridges.

After passing the Pemberton Meadows, the track has been continued to cross the Fraser at Lillooet. Here a further delay of a few weeks will ensue to permit of the building of a large bridge. The piers for this structure are already in place; and once the bridge is finished, the laying of track will continue to Clinton and above that point.

In November next, Mr. Sperry expects that construction will have advanced to such a degree of completion as to permit of the inauguration of a mixed train service from Squamish to Lillooet, a distance of 120 miles, this run being made in five or six hours. Mr. Sperry has already placed orders for a large amount of rolling stock, which is to be utilized in equipping the road for regular traffic. This equipment is to be thoroughly modern and up-to-date, so that the railway service will not be excelled by that of any other transcontinental system in Western America.

PERSONAL.

GERALD PONTON, C.E., of Calgary, has been in England during the past month investigating the various methods of road building.

ARTHUR SURVEYER, of Surveyor and Frigon, Consulting Engineers, Montreal, is leaving for Europe on August 1st, to attend the "White Coal" Congress at Lyon.

CHAS. HARPER, B.A., of the research laboratory of Queen's University, Kingston, has been appointed professor in charge of the Department of Science at Moose Jaw College.

The Canadian Northern Railway Company announces, from its Winnipeg office, the following appointments taking effect July 5th:—

Mr. I. L. Boomer, Superintendent at Edmonton, (3rd district Western), becomes Superintendent at Calgary (4th district Western Division, newly created).

Mr. J. C. O'Donnell becomes Superintendent at Edmonton, (3rd district Western), in place of Mr. Boomer. Mr. O'Donnell has been promoted from Trainmaster, his headquarters formerly being at Rainy River, Ont., on the 1st district Central.

Mr. M. G. Hurd, formerly Chief Dispatcher at Saskatoon, (2nd district Western), becomes Chief Dispatcher and Trainmaster at Calgary (4th district Western).

Mr. R. Nelson, formerly Chief Dispatcher at Edmonton, becomes Chief Dispatcher and Trainmaster at Edmonton (3rd district Western).

OBITUARY.

The death is announced of Mr. Everett Ketcheson, an assistant engineer on the construction of the Trent Valley Canal. Mr. Ketcheson was drowned in Trent River on July 11th.

From Haileybury comes the report of the death of Mr. Murdock Lloyd, mining engineer of Toronto, from injuries sustained from a boiler explosion at the Tough Oaks Mines at Swastika.

A fatality occurred near Lytton, B.C., on July 6th, when Mr. John Middleton, a member of a Canadian Northern Railway survey party was killed by falling a distance of 70 feet from a ledge of rock.

While taking measurements in a sewer tunnel Mr. Robt. Strathern, a resident engineer, Department of Sewers, city of Toronto, was asphyxiated by illuminating gas which had escaped from a broken main and had flooded the sewer. Several other members of Mr. Strathern's party were overcome and narrowly escaped death. In connection with their removal from the tunnel, and with an attempt to save the life of the resident engineer, is associated the name of Mr. M. P. McDonald, assistant engineer, whose heroic efforts have occasioned great admiration.

COMING MEETINGS.

UNION OF CANADIAN MUNICIPALITIES.—Annual Convention to be held in Sherbrooke, Que., August 3rd, 4th and 5th, 1914. Hon. Secretary, W. D. Lighthall, Westmount, Que. Assistant-Secretary, G. S. Wilson, 402 Coristine Building, Montreal.

WESTERN CANADA IRRIGATION ASSOCIATION.—Eighth Annual Meeting to be held at Penticton, B.C., on August 17, 18 and 19. Secretary, Norman S. Rankin, P.O. Box 1317, Calgary, Alta.

AMERICAN PEAT SOCIETY.—Eight Annual Meeting will be held in Duluth, Minn., on August 20th, 21st and 22nd, 1914. Secretary-Treasurer, Julius Bordollo, 17 Battery Place, New York, N.Y.

CANADIAN FORESTRY ASSOCIATION.—Annual Convention to be held in Halifax, N.S., September 1st to 4th, 1914. Secretary, James Lawler, Journal Building, Ottawa.

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

CONVENTION OF THE AMERICAN SOCIETY OF MUNICIPAL IMPROVEMENTS.—To be held in Boston, Mass., on October 6th, 7th, 8th and 9th, 1914. C. C. Brown. Indianapolis, Ind., Secretary.

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

AMERICAN ROAD BUILDERS' ASSOCIATION.—11th Annual Convention; 5th American Good Roads Congress, and 6th Annual Exhibition of Machinery and Materials. International Amphitheatre, Chicago, Ill., December 14th to 18th, 1914. Secretary, E. L. Powers, 150 Nassau St., New York, N.Y.

In *The Canadian Engineer* for July 2nd, an error appeared in reporting the name of the writer of the article entitled "Laying Outside Hill Roads," appearing on page 108. The author's name is Capt. A. C. Garner and not "Gardner," as appeared.