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NOTES ON HEADGEARS FOR COLLIERIES AND OTHER MINES.

By F. TISSINGTON.

The headgear or frame supporting the pulleys around which pass the ropes attached to the cages for the conveyance of men and material to and from the working level is generally the most important structure in any mining equipment, as on its utility and stability depends the entire output of the mine as well as the safety of the men working underground.

Under these circumstances, therefore, it will be readily seen that it is not a wise policy to attempt to skimp the material, design of details, or execution of the work in the shop, but, in view of the indeterminate nature of the vibratory stresses on the structure and the sudden application of loads, good allowances should be made for impact and low unit stresses adopted generally.

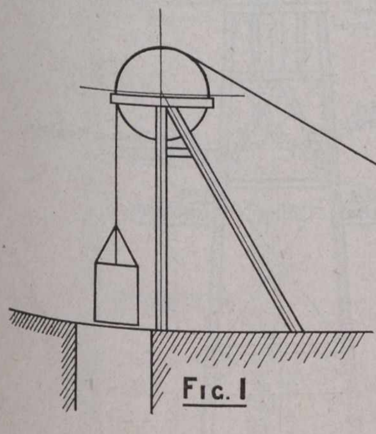


Fig. 1

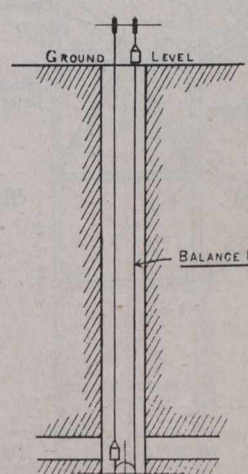


Fig. 2

Quite a large amount of the detail work is purely derived from experience and practical requirements, and this will be referred to in detail later.

Probably the earliest form of headgear consisted of a timber frame built up in the manner shown in Fig. 1. The writer has seen quite a number of this type, ranging from about fifteen to thirty feet high, with more or less detail, according to the loads carried. Generally, however, the speed of winding was very slow and the loads light; also, each frame carried one pulley only, so there was no necessity for much equipment in the way of guides.

As the seams near the surface were worked out and the demand for coal grew, a gradual evolution took place in all colliery equipment.

With the increased depth and greater time occupied in winding the material to the surface, the desire to carry more coal per trip arose, and consequently we see to-day collieries with winding plants capable of handling three and four loaded tubs per trip and landing same at the pit mouth from the working level, which may be anything up to about three parts of a mile, well under a minute.

Double, and even treble, decked cages are to-day quite an ordinary event, and occasionally you will find a plant where two pit tubs or corves are carried on each deck, each containing about 1,600 or 2,000 pounds of coal, and, with the cage, chains and rope, making in all somewhere around 30,000 to 40,000 pounds of dead weight to move on the loaded side of the shaft.

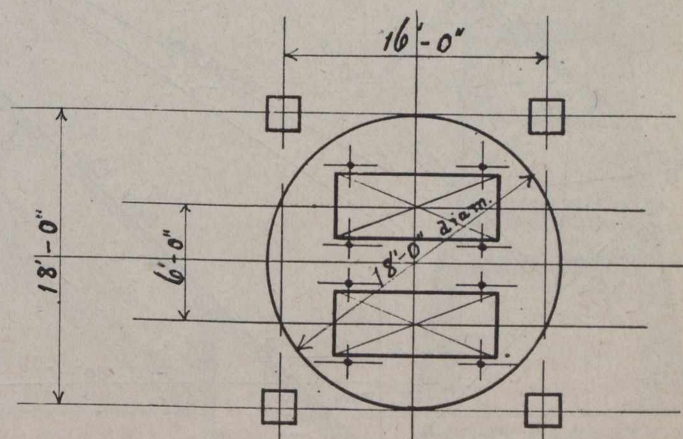


Fig. 3

Practically all modern plants are designed with two sheaves or pulleys on the headgear, each provided with a rope supporting a cage, arranged so that when one is at the top of the shaft the other is at the bottom, and thus to a large extent they balance each other, so far as the winding engine is concerned, as the two ropes are led to two drums, mounted side by side on the same shaft, one being taken over the top side of the drum on the one side, and the other led underneath the other drum, so that as soon as one rope unwinds the other winds up.

The amount of the unbalanced load is measured by the quantity of coal carried per trip and the difference in the length of the rope from the supporting pulley on the headgear to the cage.

Sometimes, however, the latter is also balanced, and this is done by attaching another rope to the bottom of each cage and passing same around a pulley in the bottom of the shaft exactly as shown in Fig. 2.

With the operation of two cages in one shaft came the necessity for the addition of guides to prevent the cages swaying about at the end of the ropes and clashing together as they passed each other.

These guides are of two general types. (1) The timber, or rail guide, which is fastened to cross pieces, called "buntons," let into the wall of the shaft at intervals to suit. These are not supported by the headgear, and, therefore, do not concern us.

(2) The other type is the guide-rope. Generally a set of these for each cage consists of four ropes, but for the larger cages six or more may be used. These are suspended directly from the headgear frame at a point just below the pulley platform. To the lower end of these ropes are attached cast iron weights in order to keep them taut and hanging vertically. Sometimes as much as ten or twelve tons will be suspended at the end of each rope, and, with the weight of the rope itself, these guides form quite a formidable load to design for. The guide-ropes, like the winding ropes, are all made of steel, and the diameter will be anything from one inch to an inch and a half.

With all winding or hoisting engines there is an increase in stress on the hoisting rope, due to the acceleration of the load, and this may amount to half the dead load suspended from the rope.

It has also been demonstrated that quite a large increase in stress is due to the vibration of the cage, caused by the pulsation of the engines, and this is very similar in effect to a piece of elastic held in the hand with a weight suspended at the free end, when a slight movement of the hand will cause the weight to jump about for a considerable period after the cause is removed. These pulsations are much more noticeable with engines of an inferior type, and also with the sudden application of the brakes or checking the speed by shutting the steam valve quickly. With an increase in load the vibrations appear to become less.

The increased stress on the rope due to acceleration may be found by the following method:—

Where P = effective pull in pounds causing motion.
T = Tension on the rope in pounds.
A = Acceleration of load in feet per second in a second.
W = Weight of cargo, load, and rope in pounds.
G = 32.

$$\text{Now } P = T - W,$$

$$\text{and also } P = \frac{WA}{G};$$

$$\text{therefore } T - W = \frac{WA}{32},$$

$$\text{or } T = W\left(1 + \frac{A}{32}\right)$$

It will be seen from this formula that if we take unit weight, then when the acceleration reaches, say, 32 feet per second, the tension in the rope will be twice that for load at rest.

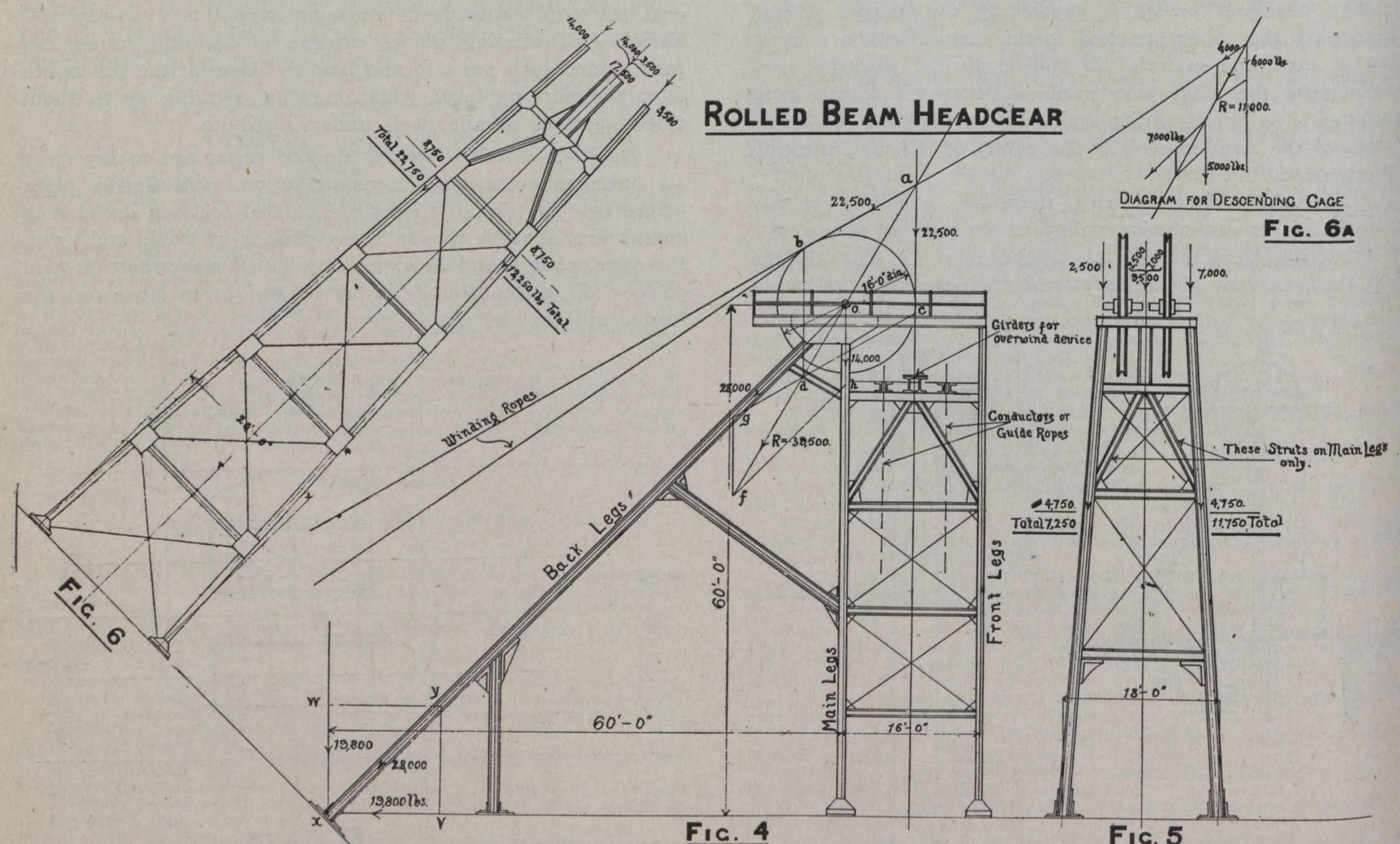


Fig. 4 Fig. 5

Therefore, knowing the acceleration of the cage and load, and also the total weight to be lifted, the tension in the rope will always be W plus some fraction of W represented by $\frac{A}{32}$. For usual practice this is generally in the neighborhood of one and a half times the load, or the acceleration is approximately sixteen feet per second in a second.

Other loads of an important character are those caused by over-winding, when a safety catch is provided for supporting the cage after the rope has been automatically released, and also the adoption of a safety catch if the rope breaks.

To the tyro it looks a most difficult operation to bring a heavy engine and its load to rest at exactly the right place every time, year in and year out, but with the appliances and safety gear used to-day it really resolves itself into a purely mechanical operation. However, mistakes are sometimes made, even with the best of men, and the winding engineman is as careful and painstaking as his brother of the railway engine.

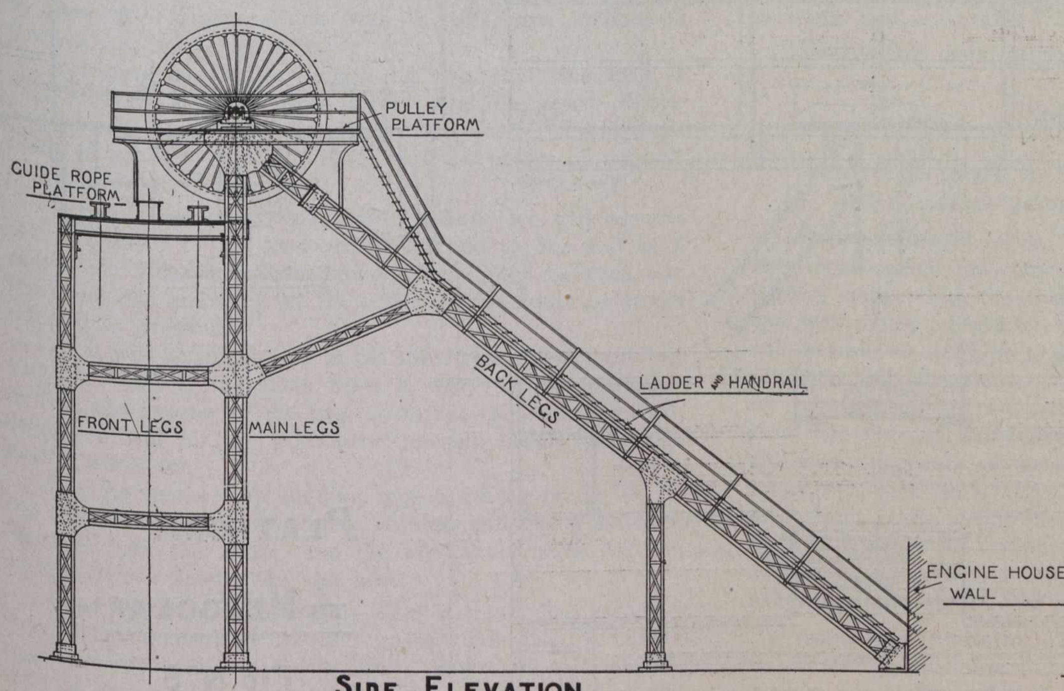
In the early days it occasionally happened that the engineman forgot to shut off his steam soon enough or omitted to apply his brakes in time, and, instead of landing his load either of men or material at the appointed place, brought the cage right over the pulley, or, failing that, broke the rope at the top of the headgear and dropped the cage to the bottom. In order to prevent this the King's safety hook was designed. This is operated by means of a heavy plate having a circular hole cut in same through which the rope passes normally, and is usually placed on the guide-rope platform. In case of an over-wind the catch, or safety hook, passes through the plate until the sides come into contact with same. This causes the rope to be released at the top, and also spreads out the part of the hook above the plate, at the same time providing a lug or catch on the safety hook which matches with the plate, and so prevents the cage from falling down the shaft again. This operation causes quite a severe shock to the headgear, as the cage and its contents would actually drop a few

(1) The rolled steel headgear made with I-beams, channels, etc., and braced with angles and tie-rods; and

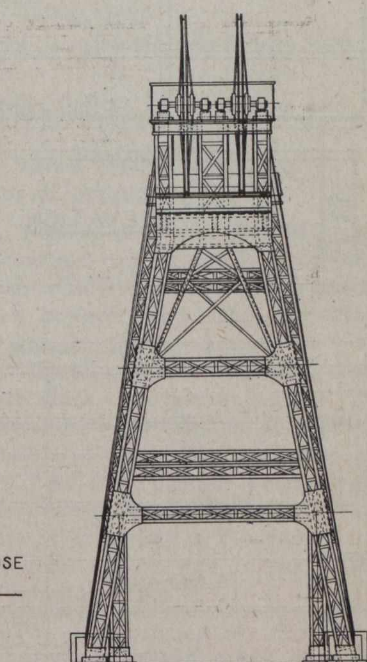
(2) The lattice headgear, in which practically the whole of the members are composed of four angles, placed to make a box section, and laced together on all four sides with small flats.

Each type possesses particular advantages. That made from rolled I-beams, etc., is easier and cheaper to produce, although the dead weight to handle will generally be greater. The facility with which the connections can be made and the use of standard details to a large extent make it rather desirable from a shop and erecting point of view, but it has one great disadvantage, and that is the lack of stiffness without the insertion of a large excess of weight, which would put it right out of the running. With the advent of the broad flange beams, however, during the last few years this has been improved to quite a large extent.

The greatest detriment to the lattice headgear is the high cost per ton produced, which, however, is offset to a large extent by the decrease in weight necessary to carry a given load, as the disposition of the material more nearly



SIDE ELEVATION



FRONT ELEVATION

FIG. No. 7

LATTICE TYPE HEADGEAR

inches, and, therefore, it is necessary in cases where this gear is adopted to allow for this extra load.

The other device, used in cases where the winding rope breaks, produces a stress on the guides, as generally it is of the slipper type, actuated by a spring, and only comes into operation by the rope parting.

The stress in this case also is liable to be very severe, as the rope may break at the maximum speed, and a small fraction of time must elapse before the various parts fulfil their duty, and during that time the cage is increasing its velocity. The action is purely of the character of a brake, and it would be fairly good practice to allow about twice the dead load of the falling cage, chains, rope and load for both cases, and also use a low unit stress on the parts affected.

In addition to the special loads described, the structure will have to be designed to carry its own dead weight, wind and snow loads. The two latter loads, however, are not of a very serious nature as compared with the live loads.

There are two types of steel headgear in general use:—

approaches the ideal form for struts and columns. Other features which prohibit its adoption in many cases are:—

(1) The increased area of metal exposed to the action of the weather, and consequently greater danger of oxidation of the structure.

(2) Painting is also more costly, not only on account of the greater area to cover, but also for the comparative inaccessibility of the various parts.

For a given amount of material there is no doubt but that the lattice structure provides a high degree of stability and freedom from vibration. Taking it all round, there appears to be very little ultimate difference in the two methods, and generally the type is chosen according to the fancy of the purchaser, or settled by the manufacturer to suit his outfit and stock material.

Figs. 4, 5 and 6 show the general outline of a rolled beam headgear, and Fig. 7 is a type drawing of a lattice structure, and from these figures a good idea may be obtained of the usual disposition of the material.

It is now proposed to give the general method of arriving at the total stress on the main members, and we will assume the following figures to form the basis of our calculations:—

	Pounds.
Weight of one cage, empty	4,000
Weight of two tubs, empty	1,000
Weight of rope	7,000
Total descending cage	12,000
Weight of coal per trip	3,000
Total ascending cage	15,000

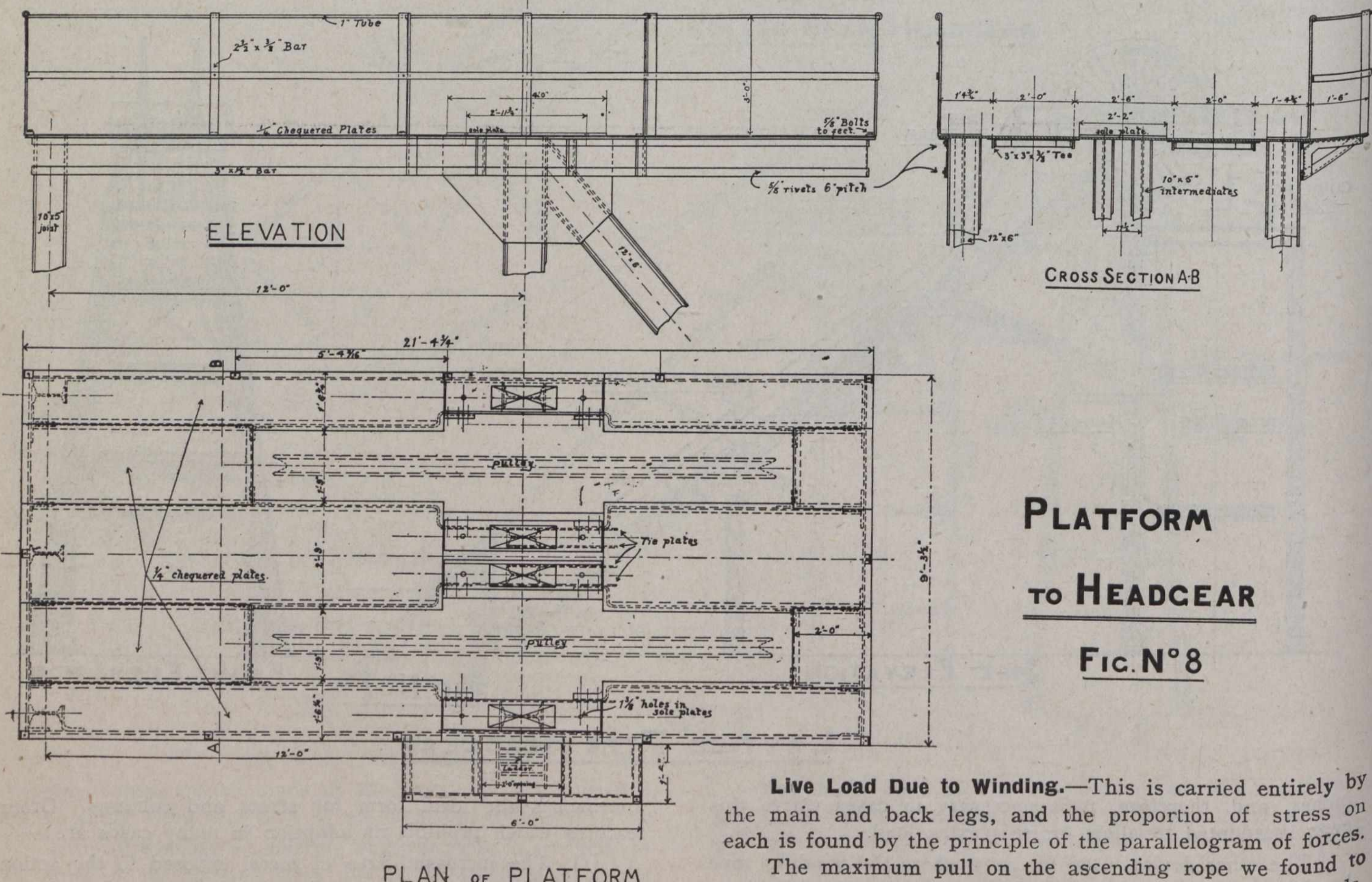
Acceleration = 16 feet per second in a second.

Therefore, stress or pull on descending rope = 6,000 pounds.

And stress on ascending rope = 22,500 pounds.

Weight of eight guide or conductor ropes and weights = 96 tons.

Height of headgear = 60 feet to centre of pulley.



PLATFORM TO HEADGEAR

FIG. No 8

Diameter of shaft = 18 feet.

In starting to set out our frame the first consideration should be to plant the legs of our structure where they will get a good, solid foundation, and also to give a diameter of pulley for the rope suitable for its size. We will, therefore, assume that it is necessary to have pulleys 16 feet diameter in order that the bending stresses do not run too high. (This information is usually supplied by purchaser.)

If this dimension is plotted down in plan, as shown in Fig. 3, it will be easy to arrive at a suitable figure for the other dimension tying up the position of the four legs round the shaft, so that the bases do not come too close to the edge of same, and it will be noted on referring to the figure that they are placed at nine feet each side of the centre line of shaft, thus making an oblong of eighteen by sixteen feet.

The position of the back legs is generally determined by the wall of the engine-house, which takes up any horizontal thrust there is. Should this support not be available for any reason, separate foundations must be made to carry the loads safely.

Referring now to Figs. 4, 5 and 6, these give the outline of the structure in side and front elevations and a true view of the back legs, respectively.

The short diagonal struts (or spurs, as they are sometimes called), shown in Fig. 4 below the girder for the over-wind gear on the guide-rope platform, are for the purpose of transferring the load from the over-wind girder directly to the main and front legs, leaving only the quiescent load on the conductors to be carried in bending.

The similar struts in the other two views also transfer the loads from the inside bearings of the two pulley shafts direct to the main and back legs without introducing transverse stress.

Having settled the general layout, we will now proceed to find the stresses on the main members.

Live Load Due to Winding.—This is carried entirely by the main and back legs, and the proportion of stress on each is found by the principle of the parallelogram of forces.

The maximum pull on the ascending rope we found to be 22,500 pounds, and for the descending one 6,000 pounds.

Referring to Fig. 4, and treating the ascending load first, if the lines indicating the rope on both sides of the pulley are produced until they meet, which will be at the point "a," and from this intersection lengths are scaled along both lines to represent the maximum tension in the ropes, as shown by "a b" and "a c," then by completing the parallelogram "a b d c" the resultant of these two forces can be found, namely, "a d," and on being measured by the same scale the magnitude of this resultant is approximately 39,500 pounds. This line, it will be noted, passes through the centre of the shaft, and indicates the desirability of using angular bearings rather than the ordinary vertical type. This resultant can now be resolved into two other components acting in the direction of the main and back legs, respectively.

Starting from the centre of the pulley shaft (which is always made the intersection point of the main and back legs) marked "o," measure along the line "o d" produced the length "o f," which represents by any suitable scale the magnitude of the resultant, viz., 39,500 pounds, and then from the point "f" draw parallel lines with the main and back legs, and the intersection of these lines with the centre lines of the two legs will complete the parallelogram "o g f h," and by the same scale with which the resultant was plotted down the magnitude of the stresses on the two legs are found to be 14,000 pounds for the main and 28,000 pounds for the back legs.

For the descending load the small diagram (Fig. 6a) gives the corresponding figures, and, referring to Figs. 5 and 6, the distribution of these stresses on the two main and two back legs will be seen, and are self-explanatory.

Guide Ropes and Weights.—There are eight of these, four to each cage, and each will weigh twelve tons, complete. Each of the main and front legs will support one-fourth of the total load, viz., 24 tons.

Stress Due to Overwind.—The maximum will be equal to the weight of the cage, tubs and coal, multiplied by two to allow for impact, which will in this case amount to 8,000 × 2 = 16,000 pounds.

Referring to Fig. 4, it will be seen that this load is supported by a heavy cross girder, and the exact distribution will depend on the relative distances between the point of application of the force and the points of support at each end of the girder.

Without having anything more definite we will assume that two-thirds of the load will be carried to one end as a maximum, and this will be equally distributed between one front and one main leg by the action of the spurs or struts referred to previously.

As it will be impossible to get the stress due to winding and over-wind at the same time, it will only be necessary to take the greater of the two, so far as the main legs are concerned, but for the front legs this will be an additional load to allow for.

For the case of the winding rope breaking in the shaft, the distribution of the stresses caused will be practically identical with the above, and the allowance made for the over-wind will answer for this also.

Dead Load of Structure.—This is generally assumed from data obtained in previous cases. For this particular case it may be taken at fifty tons. Approximately twelve tons of this will be carried by the two back legs and the remainder about equally on the main and front legs, namely, nine and a half tons each.

Wind Load.—A maximum of about 30 pounds per square foot of actual area of frame exposed will be quite good practice. As, however, it is rather difficult to obtain this information until the calculations are completed, an assumption may be made that the area exposed equals about one-fifth the total area enclosed by the outside lines of the frame.

By adopting this latter course we find the total area is about 2,800 sup. feet, giving a total wind load of 16,800 pounds.

The centre of gravity of this force will be about half-way between the ground level and centre of pulley shaft, and, therefore, the moment will be

$$16{,}800 \times 30 = 504{,}000 \text{ foot pounds.}$$

This will be divided about equally between the three pairs of back, main and front legs, and, therefore, the maximum stress on any leg will be equal to:—

$$\frac{504{,}000}{3 \times 18} = \pm 9{,}333 \text{ pounds, according to the direction of the wind.}$$

Where 18 feet = the centre of the legs at the base in feet.

Neglecting snow load, which will be comparatively small for a structure of this class, and tabulating the figures found, we have the following:—

Table of Stresses on Front, Main and Back Legs of Headgear.

Nature of stress.	Front legs.	Main legs.	Back legs.
Dead loads—			
Guide-ropes and weights..	48,000	48,000	....
Weight of structure	19,000	19,000	12,000
Total dead load	67,000	67,000	12,000
Live loads—			
Winding stress	....	14,000	28,000
Over-wind stress	2,700	2,700	....
Wind load	9,500	9,500	9,500
Maximum live load	12,200	26,200	37,500
Add 100 per cent. to live..	12,200	26,200	37,500
Grand total	91,400	119,400	87,000

The ordinary methods are now adopted for finding the sections, and a suitable formula would be:—

Permissible safe stress in pounds per square inch =

$$18{,}000 - 70\frac{L}{r}$$

Where L = length in inches between supports,
and r = least radius of gyration of section.

Pulley Platform.—Fig. 8 shows a general plan of a platform from which the construction can be clearly seen. The girders supporting the floor plates are made from angles and web plates. Sometimes, however, and where the details will permit of same, it is cheaper to use channels in place of these built-up girders. Usually, heavy steel plates, half to three-quarters of an inch thick, are laid under the cast-iron sole-plate of the pedestal with the idea of distributing the load as evenly as possible over the intersection of the main and back legs. The floor-plate should preferably be of chequer plate, and if these are judiciously arranged, quite a large amount of stiffness is imparted to the top of the structure to resist the torsional strains due to the unbalanced stresses on the two winding ropes.

Guide-rope Platform.—The construction of this would be similar to the above, and the conductor and over-wind girders would form the supports for the platform.

Practically speaking, neither platform has to support any vertical load other than that of a man walking around or any loads due to repair work going on, and the latter is not likely to be very excessive.

The design of these platforms is largely one of practical experience, and perhaps their most important function is that of a diaphragm, as suggested previously.

The pulley platform may be supported either as shown in Fig. 4 on all six legs, or as indicated in Fig. 7 on the main and back legs only, with the small angle struts inserted as steadiers.

The girders for the guide-ropes and over-wind gear are calculated for in the usual way, the only point to watch being an adequate allowance for the sudden dropping of the cage and its contents on same.

Bracings, Struts and Diagonals.—The whole of these members are put in solely for the purpose of stiffening the structure and reducing the unsupported lengths of the legs.

Generally, the working out of the details will determine to a large extent the best sections to use, and this feature, combined with the general principles of symmetry and proportion and usual practice, will bring about a result which is at once reliable and pleasing to the eye.

RESURFACING OF A TARVIA ROAD IN ST. THOMAS, ONT.

By M. H. Baker, City Engineer, St. Thomas.

In 1912 a portion of Gladstone Avenue, to the extent of 1,681 yards, was re-surfaced with Tarvia X. The portion of the street in question was one of the first macadamized streets laid in St. Thomas, having been constructed in 1894 by Mr. R. W. Campbell, then city engineer. The street provided ample material for the construction of the Tarvia pavement, with the exception of two cars of fine stone which were required for surfacing.

The Tarvia was heated in kettles and applied in the form of a spray by a small tank wagon, provided by the Paterson Manufacturing Company, steam pressure being supplied to the tank by the steam roller. This method is rather costly; the more economical method being to have the material supplied in tank cars and heated by steam.

The surface of the roadway was first swept as clean as possible with hand brooms, then spiked with the steam roller. After the roadbed was thoroughly broken up, it was found to be too dirty to apply the Tarvia, so, all the stone was screened, to remove all dust, loam and fine material. It may be of interest to note that the screenings were found to be of the following composition:—

Moisture	6%
Gravel and coarse sand	58%
Fine sand	15%
Clay and organic matter	21%

The screened stone was then graded and rolled to within two inches of the surface of the completed roadway. This surface was then given an application of Tarvia, and 1½-inch stone filled over this and rolled to approximately the surface of the completed road, and Tarvia again applied. Over this was spread stone chips to cover the Tarvia, and another application of Tarvia lightly sprayed over this. This was all covered with a coat of sand and thoroughly rolled to a smooth, hard surface. After a couple of weeks' traffic had worked the sand into the surface, the street was swept clean, and presented a smooth, hard surface.

The cost of the work was as follows:—

Tarvia	$ 408.50
Freight and cartage on Tarvia, kettles and tank wagon	111.98
Roller (a charge of $10 per day was made on the street)	105.00
Crushed stone	82.29
Miscellaneous (tools, etc.)	13.79
Labor	364.19
Sand	4.00
	$1,089.75

This makes a cost per yard of $.648.

THE SUDBURY NICKEL FIELD.

An important purchase has been negotiated in the Sudbury nickel field during the last two weeks, and is noted in the Engineering and Mining Journal. Dr. F. S. Pearson, who has been closely identified with Sir William MacKenzie in his hydro-electric power enterprises in Canada and Mexico, has taken over the holdings of the Dominion Nickel-Copper Company. It is believed, however, that Doctor Pearson and his former associates will not be called upon to do the financing, rumor having it that the money will be put up by the Rothchilds.

BRICK PAVEMENT FOUNDATIONS.

In commenting on the relative economy of using a sand or concrete foundation under a brick pavement the chief items to consider are first cost, maintenance and life. In the following discussion of the three items of first cost, maintenance and life the experience of the Department of Public Service of the City of Cleveland was described before the American Association for the Advancement of Science by Mr. Robert Hoffman, chief engineer of the Department of Public Service.

The city of Cleveland first began to lay brick pavements in 1889 and has continued doing so ever since, until at present there are about 328 miles of streets paved with brick, subdivided as follows:

257.61 miles, 5-in. brick, sand foundation.
19.39 miles, 5-in. brick, concrete foundation.
39.17 miles, 4-in. brick, concrete foundation.
11.84 miles, 4-in. brick, sand foundation.

Since then prices paid for brick pavements have varied from $1.18 per square yard to $2.48, depending on the various forms of foundation and size of brick used.

The following table indicating the various paving combinations employed in Cleveland for brick pavements shows the yearly average maximum and minimum prices paid per square yard since the year 1900. The cost of excavation assumed at 50 cents per cubic yard is included as measured below the top of the paving brick.

Cost of Brick Pavement Per Square Yard Since 1900.

Size of Brick.	Foundation.	Minimum.	Maximum.
5 in.	Natural sand	$1.18	$1.56
5 in.	8-in. sand or gravel	1.40	1.97
5 in.	6-in. concrete	1.94	2.48
4 in.	6-in. concrete	1.71	2.34
4 in.	4-in. concrete	1.47	1.73

Investigation has shown that the prices paid for the various combinations seem to rise and fall in the same years, which would indicate that the variations depended upon the material and labor market and not on the difference in combination.

The earlier of the brick pavements consisted of small blocks of fireclay 4 in. in depth, laid upon a natural sand foundation. Some of these are still in use though in service on residence streets for 20 years. In other localities, partly on account of traffic conditions, brick pavements on sand foundations required relaying in less than 15 years.

No pavements of brick laid according to recent specifications and practice, nor any laid on concrete foundations, have been in existence long enough to cause any thought of re-laying.

In reference to the brick pavements that now require re-laying having reached a condition where any adequate repair would prove far too expensive, it could probably be shown that other defects, such as improper filler, poor brick, or defective construction, had as great an influence in causing deterioration as did the sand foundation. An entirely different condition would probably be found in an open and poorly drained clay district.

In considering the relative economy of sand and concrete foundations for brick pavements experience will show that a properly laid pavement of 5-in. brick on a sand foundation will have a life of at least 15 years, if laid in residence or light business traffic streets. A 4-in. brick under similar conditions would probably have a life of three or four years less.

The problem is to compare the cost of such a pavement with one laid on a concrete foundation the actual life of which has not yet been determined by experience.

The following prices are the average for the last three years and therefore expressive of existing conditions:

5-in. brick, natural sand foundation, $1.27 per sq. yd.

5-in. brick, 8-in. sand or gravel found., $1.58 per sq. yd.

4-in. brick, 4-in. concrete foundation, $1.60 per sq. yd.

Assuming a 15-year life for the 5-in. brick pavement on a natural sand foundation, and the interest to be paid as 4 per cent., it will require a payment of 9 per cent. per year to pay interest on the cost of paving and to provide a fund for its renewal at the end of 15 years. This means an annual payment of 9 per cent. of $1.27 or $0.1143.

In order to compare this with the case of a 4-in. brick pavement laid upon 4 in. of concrete, the same annual payment of $0.1143 must be assumed. Deducting 4 per cent. of $1.60, the cost of such a pavement, for the interest charge, or $0.064, leaves $0.0503 to be applied per year for amortization. This amounts to a little over 3 per cent., and according to amortization tables would require a period of 22 years to provide the renewal fund.

In other words, the pavement upon the concrete must have a life of 22 years in order to be as cheap as the one upon the natural sand having a life of 15 years.

Comparing the cost of the 5-in. brick upon a ballast of 8 in. of sand or gravel with the brick upon 4 in. of concrete, it is readily seen that the difference is so small that both could have the same life at the same cost per year. Evidently then, the concrete foundation is as cheap as the other, and much more economical, as the probability of longer life is much greater.

A 4-in. brick laid upon a natural sand foundation would cost about $1.07 per square yard, using the same cost data as used for the 5-in. brick. If such a pavement had a life of 12 years, with money at 4 per cent., it would require an annual payment of about 11 per cent., or $0.1177.

Comparing this with 4-in. brick on 4 in. of concrete, one sees that the latter would be equally cheap per year if the life of the pavement were about 21 years. It can likewise be shown that the 5-in. brick would be as cheap as the 4-in. if the life of the former were 3 years longer.

Similar calculations will show that a 4-in. brick laid on 6 in. of concrete will be as cheap as 4 or 5-in. brick on the 8-in. sand ballast if its life is 18 years.

From this method of reasoning it may be concluded that where a natural sandy foundation under good conditions is found it will probably be as economical to lay a brick pavement without a concrete foundation as with one, and the first cost will be considerably less. In other locations, however, where it is necessary to bring upon the work from elsewhere the sand or gravel ballast, the first cost will be nearly as great, and the pavement with the concrete foundation will ultimately prove to be the more economical of the two investments.

Several other matters must, moreover, be given consideration when passing judgment on the question. In Ohio, for instance, the municipality though paying only a relatively small proportion of the cost of the initial pavement, namely, the part laid in street intersections and 2 per cent. of the remaining part, must pay 50 per cent. of the expense of relaying any pavement. It is, therefore, decidedly to the advantage of the municipality that a pavement should be so laid that its life will be as long as possible, so that the repairing expense shall come only at long intervals.

An unyielding sub-base, such as concrete affords, is highly desirable, and should be supplied wherever possible, and in most cases will prove more satisfactory even at slightly greater cost. Concrete will carry the pavement load over the many soft places caused by street openings prior to paving and will prove a greater factor of safety against settlements and irregularities liable to occur where no concrete is employed. Any settlement in a pavement foundation breaks the bond of the brick and will be rapidly followed by serious deterioration.

Another possible economy in supplying a concrete foundation may be found in the possibility that some time it may be desired to replace brick with other kinds of paving material for which a concrete foundation must be supplied, such as wood block, asphalt or asphaltic concrete, in which event the cost will be materially lessened by reason of the existing concrete.

In open country, with poor drainage facilities, there is no doubt that the damaging effect of frost and the yielding subsoil would soon depreciate any brick pavement with only a natural soil foundation, and under such conditions concrete is the only safe and economical foundation to use.

RAILWAY TERMINALS.

The subject of railway terminals is one that comes up constantly, with its increasing complexity due to the rapid growth of traffic on Canadian railways. Passenger terminals are of more immediate interest to the general public than freight terminals, and have therefore received much more attention. Yet the freight traffic on most American railways yields the greater part of the gross revenue, and arguments for greater attention to it are set forth in a paper by Mr. L. C. Fritch, chief engineer of the Chicago Great Western Railroad, presented January 14, before the Canadian Railway Club at Montreal. These notes are taken from the paper.

The cost complete of the New Grand Central Terminal in New York City will probably be $200,000,000, a sum that would build 2,000 miles of double track road at $100,000 per mile. This terminal will serve but two railroads, the New York, Central & Hudson River and the New York, New Haven & Hartford. The new Pennsylvania terminal in the same city, serving only the Pennsylvania and the Long Island Railroads, is estimated to have cost to December 31, 1910, $113,000,000. In each case the annual fixed charges, taxes and depreciation, figured at 10 per cent. of the initial cost, plus a much smaller sum for operation and maintenance, will amount to nearly half the annual gross passenger receipts at these terminals of the railroads concerned.

Similar figures may be presented for the Chicago & North Western passenger terminal in Chicago and for the terminal under construction at Kansas City, Mo. Some of the roads which will use the latter terminal will pay more for the privilege than their entire passenger receipts derived from Kansas City. The combined value of passenger terminals used by he Pennsylvania in New York, Philadelphia, Baltimore and Washington is $178,000,000. This is an average of about $800,000 per mile of road from New York to Washington, and the fixed charges alone will average $40,000 per mile.

The annual passenger revenue on all railways in the United States is about $750,000,000. The freight revenue is about $2,000,000,000, or nearly three times as much. Yet so much attention has been given to passenger terminals that only limited appropriations have been made for freight terminals, and their possibilities in the way of economies have been developed to a limited extent only. Existing facilities are in many localities grossly inadequate to handle the rapidly growing traffic with the necessary promptness, and there must be almost a revolution in such cities as Chicago in the methods so long in vogue.

PLANT INSTALLED AT THE LONDON PAPER MILLS AT DARTFORD, ENGLAND.

The plant consists of:—

1. Receiving hopper with supports placed outside the boiler-house on the bank of the river, in such a position that the coal can be deposited into the hopper by a crane and grab.

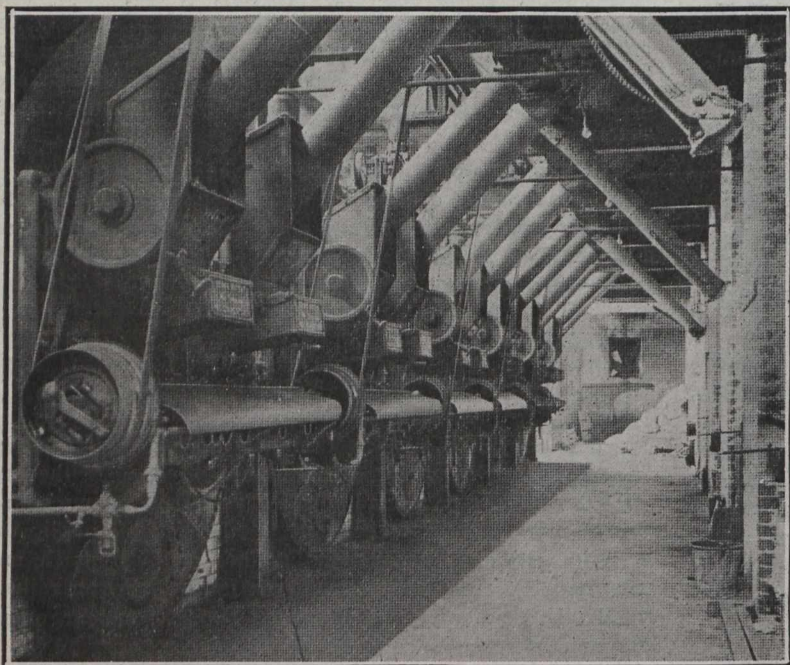


Fig. I.

A range of "Bennis High Temperature Machine Coking Stokers keeping steam in the boiler-house of the London Paper Mills.

2. Overhead coal storage bunker having a capacity of about 330 tons of coal.

3. Bennis U-link chain conveyors with driving gear and all supports.

4. Shoots to carry the fuel from the bunkers to the hoppers of the Bennis mechanical coking stokers by which the boilers are fired.

The method of operating the plant is as follows:—The coal is brought up to the wharf in barges. A grab load of coal is raised from the barge by the crane, and dumped into the receiving hopper from which it flows through an outlet into the chain conveyor, which is placed immediately beneath. The first portion of this chain conveyor is carried up an incline and the coal is thus raised to the level of the top of the bunkers. The conveyor then passes horizontally over the top of the bunkers, and the coal is dropped through openings into the storage bunkers beneath.

The coal flows by gravity from the storage bunkers into the storage hopper, and a valve is placed at the end of each shoot to control the supply of coal.

The conveyor is of the well-known U-link chain type. It has a capacity of 40 tons per hour, and is driven through suitable reduction gear by a separate motor. The overhead coal storage bunkers are built over the firing floor and are about 80′-0″ long x 11′-0″ wide having a total capacity of about 330 tons. The boiler-house wall is utilized for one side of the bunkers, the other side consists of a reinforced brickwork wall. The bottoms of the bunkers are hoppered in shape, and are formed out of rolled steel plates with suitable stiffeners and connections.

A number of rolled steel joists run the whole length of the bunker to form supports and carry the weight between the stanchions. These longitudinal joists are supported in their turn by other transverse joists spanning the firing floor and carried at one end on substantial stanchions, resting on the floor, the other end being built into the wall of the boiler-house. These stanchions are also used for supporting one side of the main boiler-house roof, and a light roof is built over the top of the bunkers themselves.

Access is given to the conveyor chamber over the bunkers by ladders and gangways, arranged so that all running parts are easy of access for inspection and lubrication.

The bunker was designed to take full advantage of the existing buildings and to give the required storage without forming any obstruction to the movements of the boiler attendant. It is so arranged that future extensions can be easily effected.

The work of the U-link chain conveyors is to feed the coal to the bunkers which deliver it by means of openings into the distributing shoots which deposit it in the "Bennis" high temperature smokeless and gritless coking stokers. The illustration affords a good view of this range of stokers with the shoots in position and in close connection with the bunkers immediately above.

It is, of course, important that steam-raising in paper mills should be smokeless and gritless. Harm will be done to both paper and pulp in various stages of their manufacture should grits and smuts be emitted from the chimney and so find their way into the incomplete manufactures.

The plant was designed and installed by Edward Bennis and Company, Limited, who are represented in Canada by Geo. H. Tod, of Toronto.

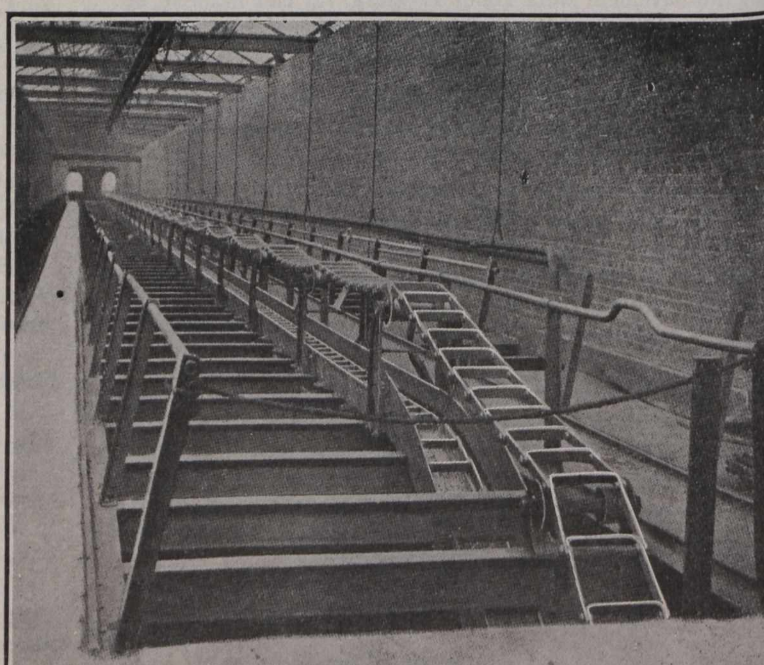


Fig. II.

"Bennis" U-link Chain Conveyor which in part of its travel is inclined to carry the coal to the level of the top of the bunkers. It deposits its contents by means of openings into the storage bunkers beneath. It forms part of the "Bennis" installation of coal handling plant in the boiler-house of the London Paper Mills.

REINFORCED CONCRETE IN CHURCHES.

By V. J. Elmont, C.E., A.M.Can.Soc.C.E.

The essential requirements that one must meet in designing churches and similar buildings are that they should be absolute fireproof and have proper acoustic qualities; these requirements are best met by reinforced concrete. In respect of this latter it is interesting to note that organs have actually been built of reinforced concrete in the United States.

Fig. 1.

Its ability to resist fire has been tested over and over again. Numerous fires in actual buildings have shown that concrete is practically impervious to fire. Also the various technical laboratories have proved how slowly heat penetrates concrete and the fitness of reinforced concrete structures to resist high temperatures. The worst that the ordinarily severe fire will do is to cause a crumbling of concrete on surfaces exposed to the heat, rarely to a depth of more than one inch. Both on this and on the other side of the ocean a great number of test buildings have been constructed entirely in reinforced concrete in order to determine the varying importance of the intensity and duration of the heat applied, its method of application, and the thickness of protective covering.

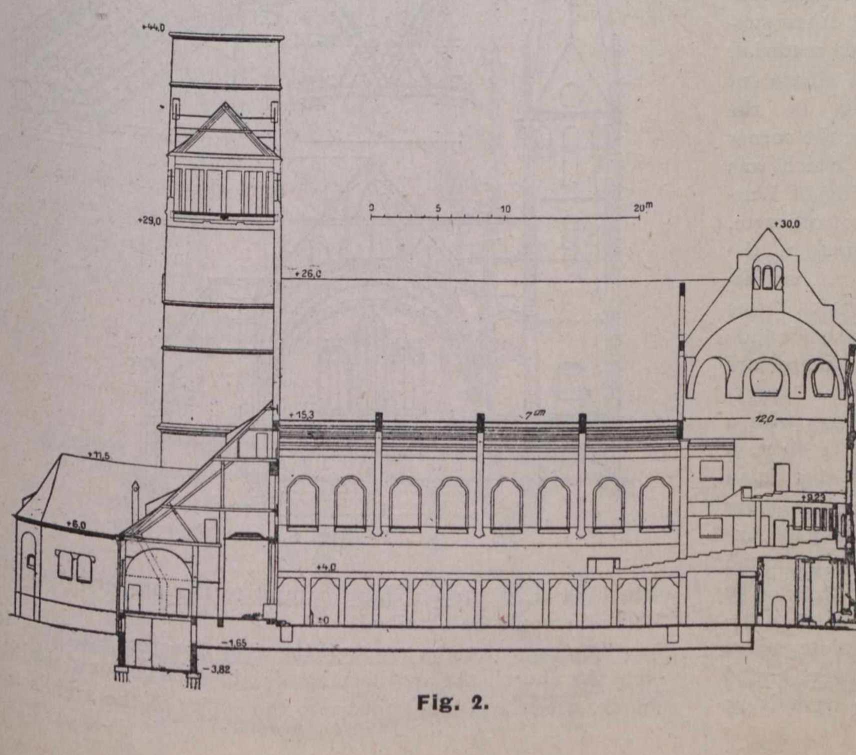


Fig. 2.

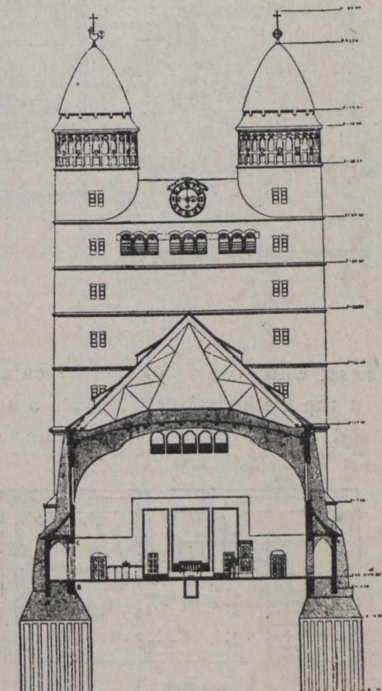


Fig. 3.

As seen from an engineer's point of view, reinforced concrete solves in the easiest and most economical way the difficult problems of construction that constantly occur in the class of buildings we are considering. Reinforced concrete renders it possible also for the architect to design with a free hand and only to take the artistic requirements into consideration. Where steel is used many complications arise through the use of complex construction and also much time is lost and cost entailed in carrying out such work, not to mention the expenses for the covering of such steel with a fireproof material.

The elementary forms which go to make the skeleton usually found in churches, are floors, columns, cantilevers, arches, roofs often with very long spans, consoles, and domes. As an expert in the proper use of adopting reinforced concrete to church building might be mentioned Professor Fisher, in Munich. He has not only used this material in the construction of the skeleton of the new garrison church in Ulm, but given it equal prominence with natural masonry and brickwork, and it appears visible in the elevations; this is one of the first examples we have of this material being used in such a way in large European buildings of an ecclesiastical type. The aforementioned architect obtains without any great expense in this church a span

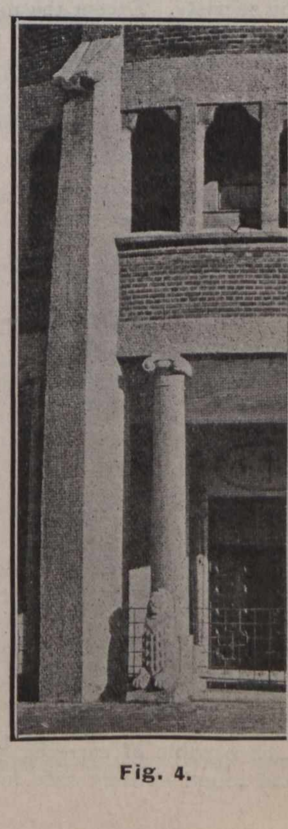


Fig. 4.

which has never been exceeded in any other church of solid masonry. In our medieval cathedrals there are very few spans above 50 feet; the renaissance builders were the first as opposed to the gothic to strive after an effect of spaciousness, which was obtained by enlarging the main dimensions. In the Cathedral of Florence a span of 56 feet was obtained and at St. Peter's, in Rome, the maximum 75 feet was reached. Considering the cost and time which only some few decades ago were necessary to solve similar problems, it will be at once seen how reinforced concrete has accelerated architectural progress.

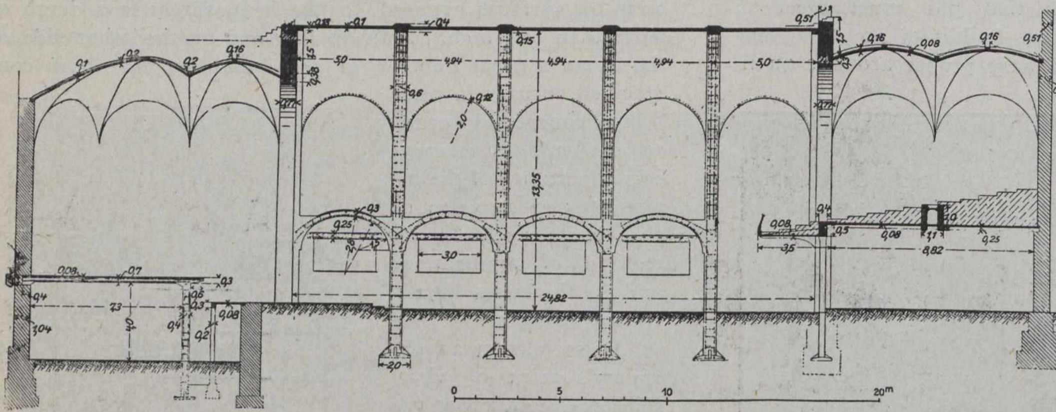


Fig. 5.

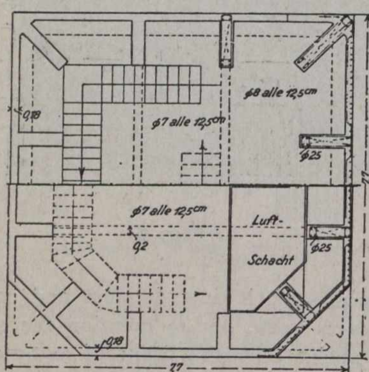


Fig. 7.

Fig. 1 shows the exterior of the church with the concrete parts easily recognizable in the buttresses and in the organ wing. The church consists of three main parts, the nave proper, the organ wing and the two towers with connecting walls containing classrooms for confirmation and the sacristy. Except the towers, in which only the floors and the main cornice are built in reinforced concrete, all the other supports are of this material. There is not a single cut stone existing in the whole building, the corner "stone" also, which was laid by the King of Wurtemberg, was of concrete. The construction in the organ wing does not differ very much from forms in common use outside the fact that there are some cantilevers carrying the galleries with a free span of 25 feet, a roof with 40 feet span and a dome. The more interesting points of construction are to be found in the nave (Fig. 2). The foundation consists of 35-foot-long concrete piles, each capable of carrying 55,000 pounds. These were tested and found entirely safe for 85,000 pounds. The nave is 75 feet broad, 100 feet long and 30 feet high and is built up of four enormous girders in arch form. (Fig. 3). Between these spans a slab and beam ceiling forms the vault over the church. The distance between the girders, which are 1½ feet in width, is 25 feet, and in designing these the architect only took the architectural requirements for a good design into consideration, not being bound in any way by the material.

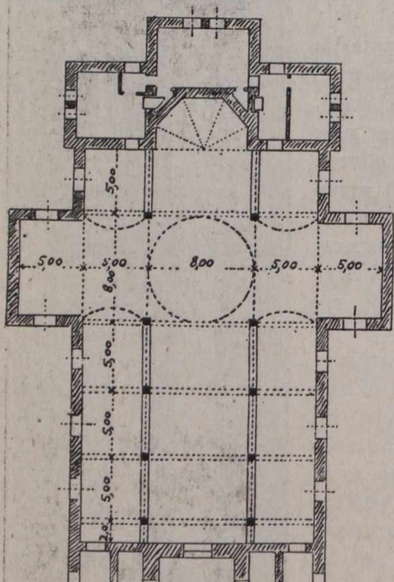


Fig. 9.

Running lengthways in the side walls of the nave (Fig. 3) are reinforced concrete beams supported by the columns, partly forming the plinth and cornice and partly carrying the brickwork filling which is 12 inches thick.

All concrete surfaces are unplastered, but treated as cut stones and bush hammered. Fig. 4 shows a part of the exterior of the organ wing. The special concrete mixture, used in the exterior, consists of 1 cement, 1 sand and 2 gravel of pea size.

By using a special mixture and afterwards washing and brushing it, a surface more natural to the properties of the concrete would be obtained.

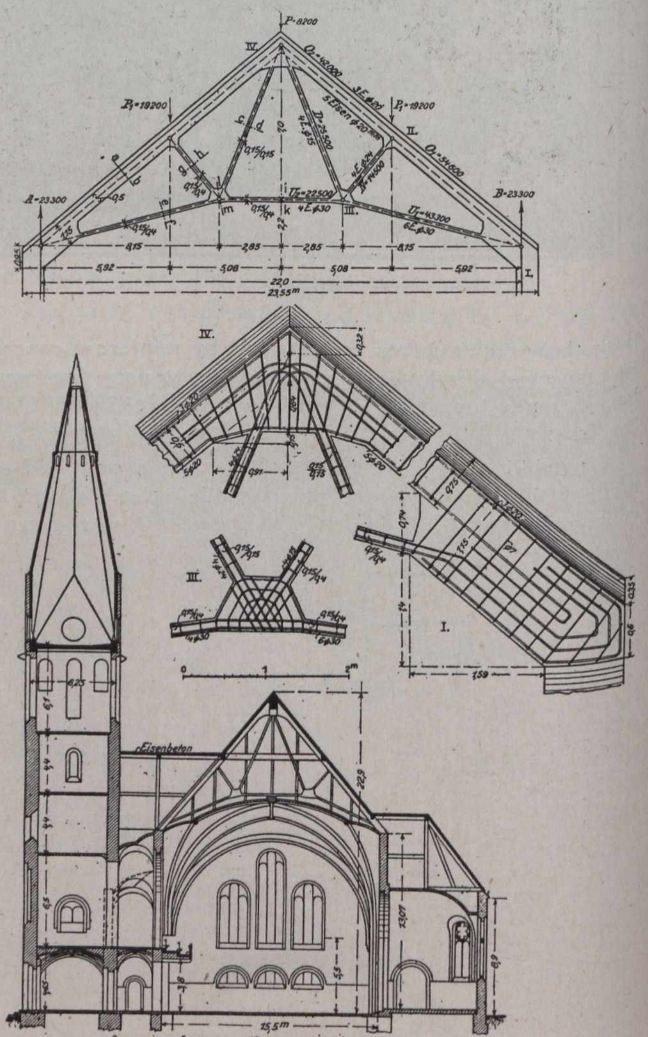


Fig. 8.

For relieving the monotonous grey concrete surfaces inside the church the architect has inlaid small colored (white, black, green and blue) tiles into the soffits of the arches and

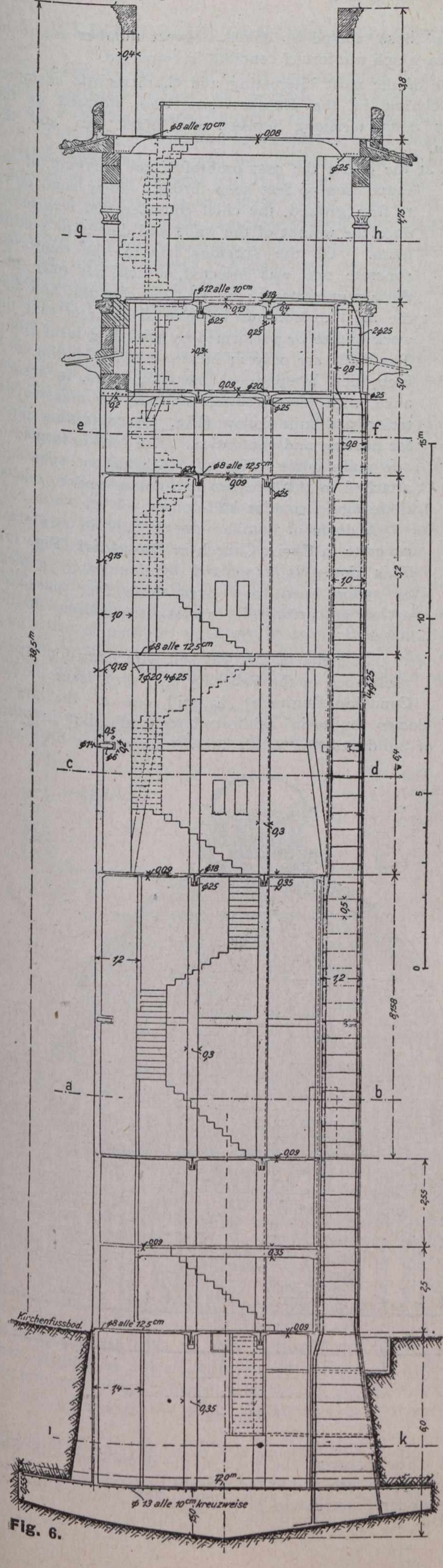


Fig. 6.

the beams. The placing of these hard-burnt glazed tiles to the concrete has produced a very good effect; such a ceramic embellishment may perhaps be developed further and in other positions, also to good effect.

In the Markus church, in Stuttgart, the nave is also built up of reinforced concrete arch girders. It is 80 feet long, 50

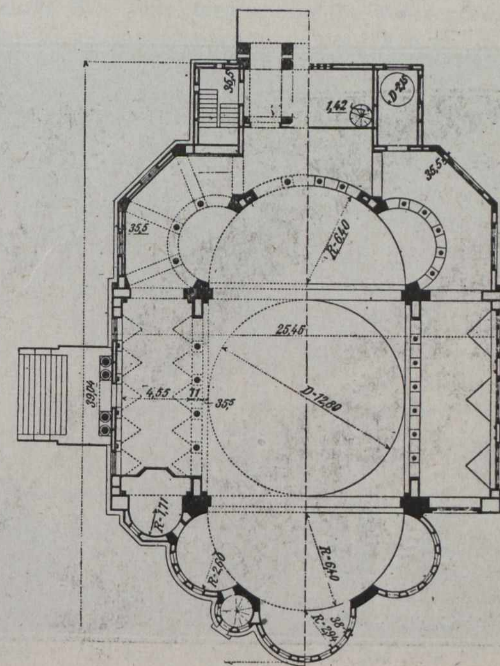


Fig. 10.

feet broad and 45 feet high. Attached to it are two side naves 9 feet broad and 15 feet high. The arch over the main nave consists of the said arch girders, spaced 16 feet centre to centre (Fig. 5) connected by a 4-inch-thick concrete slab. As the architect only permitted 6 inches of the girders to be visible under the slab the rest of the thickness was was placed over the slab. Downward, the arch girders go over into the columns, in which the beams and columns for the side

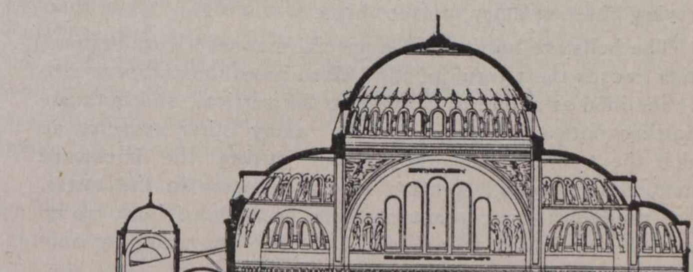


Fig. 11.

naves are also fixed, so that the frame for the three naves forms a monolithic, connected whole, of great strength and stiffness.

An interesting feature in this church is that the 190-foot-high tower is built up in reinforced concrete. After careful examination and deliberation the leading consultants for this church building decided upon the use of this material, apart from any question of economy, being of the opinion that its employment minimized the possibility of cracks arising from the swinging of the bells to a greater degree than in any other material, and also that it offered the greatest resistance in proportion to its weight, and was capable of resisting tensile as well as compression stresses, and in addition to these advantages was perfectly fireproof.

The tower (Figs. 6 and 7) rests on 12 reinforced concrete columns connected with 8-inch-thick concrete walls; they go along the total height of the tower and are at the base 1 foot by 7 feet, and at the top 1 foot by 2½ feet. They are stiffened laterally by the floors. The foundation slab, 40 feet square, is situated 20 feet under the church floor, from this slab and up to a height of 60 feet the tower is 25 feet by 25 feet. From that elevation the corners are taken off so that it continues as an octagon. In elevation 90 feet above the floor the reinforced concrete walls stop short and are continued in brick. Over the room for the bells are only four columns carrying the last story, 16 feet high.

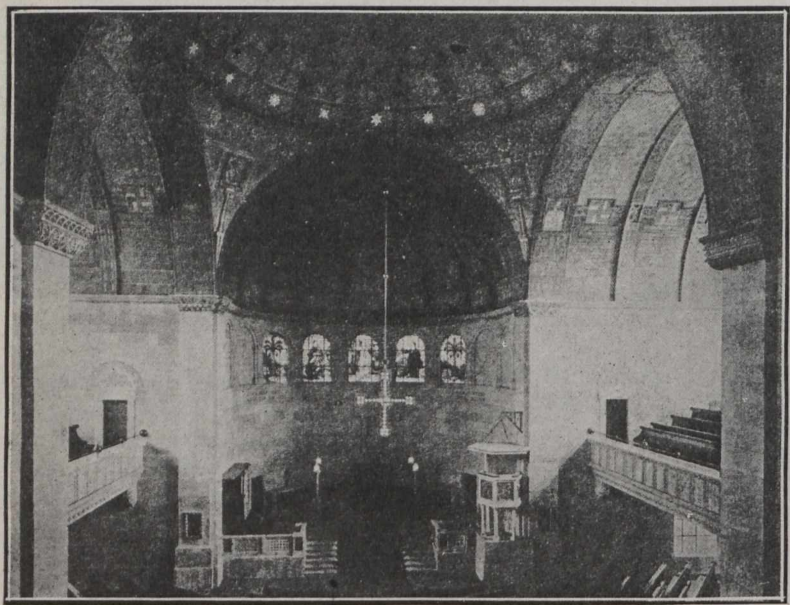


Fig. 12.

The bells are hung up in a special manner (Kurz system) which causes the tongue of the bell to meet the clapper when it is inclined at about 45 degrees to the vertical, and both are swinging outwards as opposed to many other systems, in which the tongue meets the clapper during the backward swing, thus causing greater impact stresses in the tower. After the Kurz system church bells have been built up to 13,000 lbs. weight, and are capable of being rung by only two men.

In the garrison church in Kiel the roof, the top of the tower and the gallery pews are constructed in reinforced concrete. The roof (Fig. 8) is supported by two diagonal Polonceau trusses of 75-foot span, built entirely of concrete and reinforced with thin bars. The tower top, placed upon the 85-foot-high brick tower, has 4-inch-thick walls, is 21 feet square at the base and 65 feet high.

In Russia reinforced concrete has already been in use for over 10 years in church buildings, not only in the greater cities but even in the villages, where one would think that the very low temperature during the long winter, the difficulty of transport and of getting the necessary able labors would throw insuperable obstacles in the way. This view (Fig. 9) shows the typical plan of such a village church and with the interior entirely built up in reinforced concrete. The roof slab is supported by concrete beams with 26-foot span between the columns and 16 feet between the brick walls and columns. Not only in those churches roofed by arches but also in those roofed by domes a great number are to be found in which reinforced concrete is employed.

One of the most interesting and characteristic examples is the church in the Russian city, Poti, situated in the swampy delta territory between the Black Sea and the Caspian.

As at the outset the cost prohibited the carrying of the foundation 70 feet deep through shifty sand down to firm ground, the chief consideration was to reduce the weight of the walls and domes to a minimum. Of the schemes sent in, the reinforced concrete one was selected, which not only met these requirements, but was the cheapest and also the most expeditious in spite of the fact that the work had to be performed by unskilled local labor. The walls are only 14 inches thick, including a 7-inch-thick airspace. The main piers, in accordance with the architect's desire for massive exterior, are built hollow (Fig. 10) containing pipes for heating and ventilation. The roof is formed of one main dome (Fig. 11) with 42-foot span and attached to this are two half-domes and to one of these again smaller half-domes.

A dome of similar size is built in reinforced concrete in Christ Church in Dusseldorf (Fig. 12). This dome is supported by four arch girders of 45-foot span, and over these four parabolic arches are arranged, carrying the weight of the tower.

The dome in Los Angeles, with its 62-foot span, (described in the February 6th, 1913, issue of The Canadian Engineer) is still one of the largest church domes built in reinforced concrete, but in other classes of buildings this width has in many cases been surpassed. For instance, the span of the dome shown in Fig. 13 is nearly 100 feet, and at the present time one with 200 feet span is under construction.

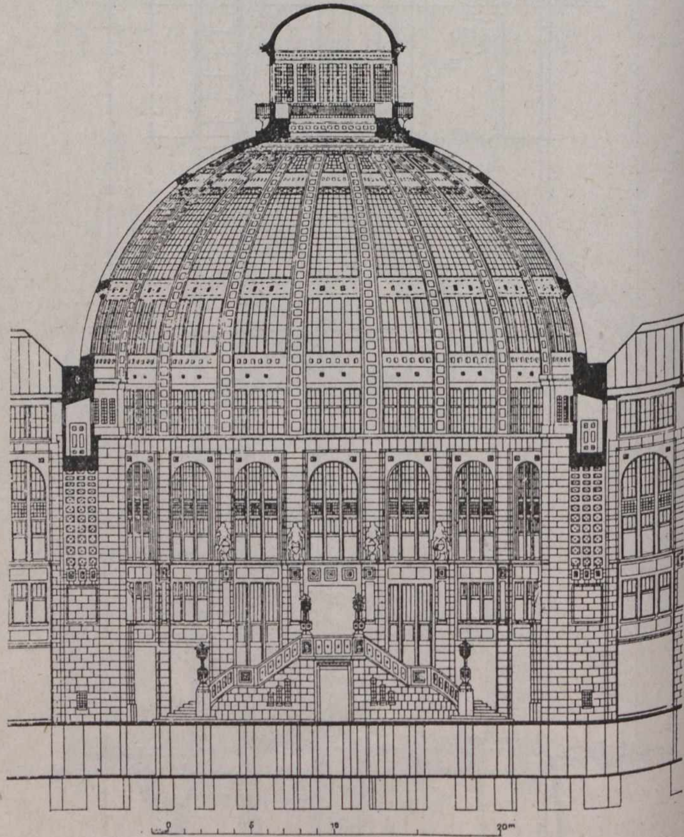


Fig. 13.

A LARGE REINFORCED CONCRETE STANDPIPE.

The details of the design of the 300,000-gallon reinforced concrete standpipe which was recently constructed in the town of Penetanguishene, Ontario, are shown herewith in Fig. 1. This tank is described in a recent issue of Engineering and Contracting. As will be noted from the drawings, the tank is 50 ft. in diameter and 21 ft. deep. The side walls are of 1:1:2 concrete, 12 ins. thick at the base and 8 ins. thick at the top. The walls are made thicker than necessary for strength in order to prevent the formation of a thick ice crust. The tank is covered by a reinforced concrete dome of a height of 1/10 of the diameter. It is 4 ins. thick and is reinforced by ⅜-in. bars 12-in. on centres.

The tank was built in about six weeks during October and the early portion of November of 1912. It was filled the latter part of December, and did not show a leak or sweating at the first filling nor thereafter.

The reinforcement of the shell was figured by a method not ordinarily used in the United States. If we consider a shell alone and assume that it is not connected with the bottom, it will increase in size as shown by curve A of Fig. 2. Inasmuch as the shell is connected with the bottom, and besides rests on the ground, it cannot elongate at the bottom, and if a proper connection is made between the side walls and bottom, the lowest portion of the shell cannot even change its directions at the bottom; or, in other words, it is fixed at the bottom. Hence, the real deformation of the shell will be a line somewhere as shown by curve B in Fig. 2. It clearly depends on the thickness and height of the shell where the deviation from the ideal line of deformation stops.

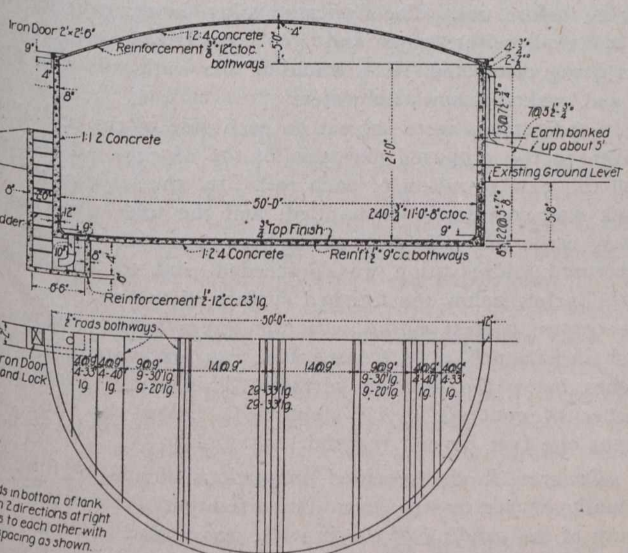


Fig. 1.—Details of Design of New 300,000-Gallon Reinforced Concrete Standpipe for Penetanguishene, Ontario, Waterworks.

This problem was first investigated by Professor Grashof, and published in his book on "Theory of Elasticity" in 1878. The differential equations governing the conditions are, however, of a high order, and even in the simplest case where the walls are of uniform thickness, the equations for the elastic curve are expressed in periodical functions and it takes several days' labor to solve a single problem. The equation cannot be solved for walls of various thicknesses, as the integrals of the differential equations are unknown up to this day. However, the elastic equations clearly show that the shape of the elastic curve is a function of $\frac{h^2}{rt}$, wherein h is the height of the tank, r the radius of the tank, and t the thickness of the shell at the base, all being expressed, of course, in the same unit.

In the present case $\frac{h^2}{rt}$ equals 16 and the elastic curve for this case only starts to deviate abruptly from the ideal deformation at the point of 4/10 h above the base, as shown in Fig. 2, curve B. This means that the water pressure corresponding to the shaded portions is not taken up by the ring action but by the cantilever action of the connection of base and shell. After a little consideration, it will be clear that there must be acting on the ideal beam, for this case a force above the 4/10 h point, which tends to bend it back into the

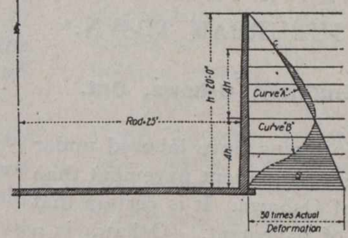


Fig. 2. Deformation Diagram.

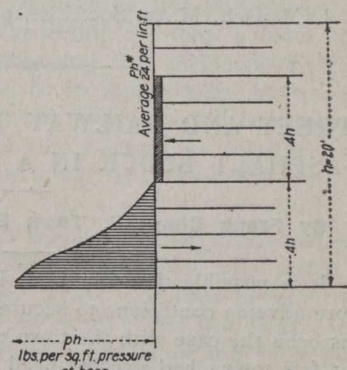


Fig. 3. Loading on Ideal Beam.

line formed by the ideal deformation of the ring sections, as the cantilever would tend to bend the top portion further out and the deformation on top must be zero from the nature of the case. This force is nearly a uniform load also for a height of 4/10 h in this case and equals approximately $\frac{ph}{24}$ in this case. Now, if a beam is assumed which is acted upon by the forces as shown in Fig. 3, we can find the elastic curve from the equation $\frac{d^2y}{dx^2} = -\frac{M}{EI}$, wherein M is the moment at any section x feet from the top, E the modulus of elasticity and I the moment of inertia at the section x. If the assumption of the form and position of curve B of Fig. 2 is correct, the elastic curve obtained by the foregoing general equation must be identical with curve B. This agreement can be reached after a few trials.

It is also clear that there exists beside the negative moment at the bottom of the shell on the inside, a positive moment higher up on the outside of the shell. The maximum value of this positive moment occurs where the shear passes through the zero value. The shear at the bottom of the shell can also be obtained from Fig. 3. This shearing stress governs the reinforcing of the bottom of the tank. The moment at the bottom of the shell, in this case is about

$$\frac{ph^3}{66} = \frac{62.5 \times 20^3}{66} = 7{,}600 \text{ ft.-lbs. per lin. ft.}$$

The reinforcing here adopted is more than ample. The positive moment on the outside of the shell is $\frac{ph^3}{217}$, which can be taken up by the concrete without reinforcing. If the reinforcement at the bottom of the shell be omitted, the tank will first crack on the inside of the shell at the junction with the bottom, and then a larger bending moment will appear on the outside of the tank $= \frac{ph^3}{163}$ in this case. This bending moment is already large enough to cause cracking of the concrete at the surface of separation between the portions placed on different days.

For other values of $\frac{h^2}{rt}$ the shape of the elastic curve is, of course, different from that here shown as curve B of Fig. 2. The greater the value of $\frac{h^2}{rt}$ the lower the point where the curve deviates from the ideal deformation of the ring sections.

The standpipe here described was designed and built by L. J. Mensch, M.Am.Soc.C.E., of Chicago.

STREET AND RAILWAY TRACK PAVING WITH ASPHALT BLOCK IN A SUBURBAN TOWN.*

By Frank Chappell, Town Engineer, Oshawa, Ont.

It is possible that few Ontario towns have labored under more adverse conditions to secure a permanent pavement than has been the case with the town of Oshawa. It is certain that but few towns had greater need of pavement. Oshawa is a thriving manufacturing town, 34 miles east of Toronto and within four miles of Lake Ontario. The centre of the town lies in a hollow in what is said was formerly a cedar swamp. While excavating for a catch basin, a row of logs side by side—a relic of an old corduroy road—was found, six feet below the surface of the present street. This fact alone would suggest the difficult nature of the soil to be contended with.

Then again, the street railway company in Oshawa, operate under a Dominion Government Charter granted them about seventeen years ago. By reason of the looseness of agreements and by-laws drawn up between them and the town at that time, the street railway company are not obliged to pave between their tracks or pay any proportion of the cost of such paving. This refusal of the company to recognize any obligation or necessity on their part of paving their right of way, was the greatest stumbling block. Special legislation was at last secured, however, permitting the corporation to finance the whole work as a local improvement, the railway company agreeing to relay its track to conform to the new grade established, and further agreeing to pay a portion (amounting to only about 65 per cent.), of the extra cost incurred in placing concrete under the ties. This percentage, moreover, was to be made payable over a period of 20 years. It might be well to state here that railway companies, making overtures to pass over the streets of Oshawa now, are not allowed to slip easily through, but on the contrary have to face and commit themselves to the most binding legal documents it is possible to execute.

The selection of a pavement had again and again been discussed. One year the cheapness of construction of Macadam road proved an attraction, but after a short strip had been constructed upon one of the main residential streets, it was found to be so dusty in summer and so muddy in the spring and fall of the year as to preclude its usefullness in the business portion of the town. The desirability of a plain concrete pavement was then discussed, mainly because of its reported cheapness. The poor economy of these cheap ideas are the biggest drawbacks to communities in the transitory stage from village to town, and from town to city. While the concrete pavement in this instance would have been an interesting experiment, it was, however, voted down by the people immediately concerned.

The asphalt block pavement upon a concrete base was finally chosen after the mature deliberation of all parties. A sheet asphalt pavement is beyond the reach of a small town because of the expensive plant necessary at all times to handle the construction or repairs which might be required. Each asphalt block, however, may be regarded as an asphalt pavement in miniature, its composition and manufacture approaching to a considerable extent that of sheet asphalt. When well laid, the blocks, while possessing many of the desirable qualities lack one extremely undesirable feature of sheet asphalt, namely local creeping.

The streets to be paved in Oshawa, were the two main thoroughfares, which intersect each other at right angles in the centre of the town. The street railway track lies along both streets. In order that the contractors might have every facility for carrying their work to a successful issue, the work was divided into four blocks or sections, and all adjacent street intersections were closed to traffic while any particular section was being constructed. In each case the work was started at the extreme end of a section and progress was made towards the centre—known locally as the four corners.

The traffic was thus but little inconvenienced, for while the last portion was being completed in the centre, the other streets and intersections were open for traffic. Moreover, by closing the street against traffic of any kind, it was possible to make a more satisfactory job of the track work, as well as giving the road a greater period for setting and seasoning before use. The first step taken in construction was to make all sewer, water and gas connections, even vacant lots being connected. The whole of this work was well filled in and soaked down with water.

Grade stakes were set out on each side of the street and driven to the proposed elevation of top of curb. This was also the new elevation of each rail. In the meantime the track was completely dismantled, and the old ties hauled away, while the rails and angle bars were thrown on the boulevard. Excavation was proceeded with to a sub-grade eight inches below the finished surface of the pavement. In the centre, corresponding with the street railway allowance a shallow trench, 9 ft. 6 ins. wide, was excavated eighteen inches below the finished surface. This would allow eight inches of concrete to be deposited below the ties, and about one foot beyond the ends.

Concrete in the specified proportions of one to nine, but actually nearer one to seven (an alteration necessary on account of the grading of the gravel), was placed in this trench to a depth of six inches. Within twenty-four hours the ties were laid on this and after the rails had been spiked and jointed, were brought to grade with wooden shims, about three or four to the rail. This left a two inch space to be filled with concrete. Great care had of necessity to be observed, in order to pack the concrete under the ties, and a very wet concrete was used, entirely free from large stones. This was done as it was believed that the ties would be more securely bedded in the concrete than if the whole eight inches had been filled at once. Furthermore the track was more easily and accurately brought to grade with this two inches of play.

The method of laying the ties and rails and subsequently raising to grade on the first six inch layer of concrete is avoided by contractors because of traversing the work twice with concrete mixers, etc. In over a mile of similar work done later this method was insisted upon, however, by the writer, and is still believed to be a far better plan than to lay the concrete all at once. The rails and ties if graded up to their

*Paper presented to the Canadian Society of Civil Engineers, February 6, 1913.

proper position are so easily disturbed by gravel teams, wheel barrows, etc., while a much longer period is necessary for the setting of such a thickness of concrete—longer than is permissible in street paving work. The second application of concrete was made at the same time that the base was being laid for both sides of the road. The curbs were also prepared at the same time, the base and sides thus being practically monolithic.

As far as track construction is concerned it is not pretended for one moment that the work described in this paper is by any means ideal. As noted in the preamble, the corporation was largely at the mercy of the railway company, which refused to admit any obligation. So when the company insisted on using old 56-lb. T. rails, 4" deep, which had been in constant use for nearly twenty years, the only thing to be done was to make the best of such unfavorable material. New ties were provided of Norway pine, but the old angle bars were used. This in particular was a point to which the writer was strongly opposed. For such rails, the writer recommended that the ends of the old rails be cut off and that they be redrilled.

Moreover, a continuous bar joint was to be used. The reason for this was, that in using the old rails and old bars after they had been once dismantled, different bars would probably be connecting other joints. Now these bars during their long service, some 15 to 20 years, would have acquired a "set" in some direction, and this could certainly not be taken out by merely tightening the bolts in re-jointing. However, this was over-ruled by the company, and the only pieces of track replaced with new material were a few split rails and defective switches. The fallacy of expecting a miscellaneous collection of angle bars to fit an equally miscellaneous quantity of defective rails was amply proven in less than six months after the work was completed.

The concrete base was well laid and is an excellent piece of work. It is possible now, however, in less than a year, to stand on a rail joint and cause a perceptible vibration with one's foot. The effect can be imagined, therefore, of street cars hammering on such joints. Even the railroad authorities now admit the poor economy shown in such parsimonious construction, and later work has been done with heavier rails having newly cut ends and new bars, although the continuous bar-joint which the writer believes is absolutely necessary in street railway work is still kept out, only, however, on the score of extra expense. The ties were laid at two foot centres with additional ties thrust under the joints. With the second layer of concrete, the ties became perfectly embedded at the ends, sides, and over the top.

The whole base was now ready for block laying and this took place usually within about three days of the concrete base being laid. A slightly dampened mixture of cement and sand in the proportions of 1 to 3 was distributed over the concrete surface to a uniform thickness of one-half an inch. Between the rails allowance was made so that a crown of half an inch could be given the blocks. The blocks were laid upon this cushion of cement and sand, and then rolled with a heavy hand roller of from three to four tons weight. The surface was then minutely inspected for any imperfectly bedded blocks, which were at once removed with specially shaped tongs, and a small portion of cement cushion placed so that the block would not rock. The blocks were laid between the rails in the same line as the rail. Now this is a nice looking piece of construction and it also has the advantage of being very easy to lay, and furthermore the number of "bats," or broken blocks, is considerably reduced.

The writer has noticed, however, that where cars have been derailed, the tendency is to open up the unbroken course for a considerable distance, should the car wheel strike the longitudinal joint. It is a question that would be well to note, as to whether any greater advantage is gained over this method by the more expensive work of cutting and fitting blocks within the track, at right angles to the line of traffic.

The fact that the blocks were laid in the same line as the rail is another regrettable instance of the overruling of the professional man by the lay man. Two and one-half inch blocks are too small for railroad work in the first place. The street car wheels abrade a place for themselves assisted by external traffic to a depth of about one and one-half inches. This then leaves but one inch of block to resist any lateral pressure which may be brought to bear by vehicles crossing a street diagonally. The result is that a block becomes loosened when laid in the same line as the rail by reason of the open surface offered by its longer side. As soon as one block is thrown out it is less difficult to loosen others. If the blocks had been laid in this instance as specified, at right angles to the rail, there would have been at least greater holding surface.

After all blocks had been rolled, examined and found to "bed" properly, the surface was sprayed with water from a fine nozzle. A cement grout was then prepared in equal proportions of cement and sand. This was poured over the blocks and brushed well into all joints and interstices. As this grout dried out it settled until the joints were only about three parts full. A second application of a stronger grout was then applied, two parts cement to one part sand, and this mixture was brushed into the joints until they were packed full. As the rails in the track were T rails, a space about two inches wide was left on the inside of each rail to be worked into a groove for the flanges of the car wheels.

A concrete of crushed granite and cement was packed well into this space and shaped with a tool to the required width and depth. This has proven a very satisfactory piece of work and while undoubtedly cheaper, it is in the opinion of the writer, under certain conditions even to be preferred to scoria blocks and granite "setts." The car wheels crush and abrade the concrete and the edges of the blocks to an extent necessary for clearance and after a little wear, a smooth uniform groove is made. The blocks are soft enough to yield to the abrading action of the car wheels without loosening the joints, while the little inequalities in the concrete groove are subject to direct crushing.

With reference to the concrete groove between the blocks and the rail, this too would probably have shown to greater advantage had a thicker block been used, say 3" or 4". This, of course, was not possible on account of the 4" rail in use. There is this about the concrete groove, however, that when abrasion occurs the dust that is produced washes away and the groove has that much less material. To overcome this the writer has since tried on over a mile of road, a groove filler of a mixture of bitumen and broken stone. This is almost a ductile composition and it is possible that in warm weather, at any rate, the bituminous mass will accommodate itself to the groove, and give better results than similarly placed cement.

In connection with turnouts and switches, the triangular space facing the frog gave considerable difficulty. Blocks were in the first place cut and fitted and grouted in. The space was so narrow, however, that the wheels would push them out of place. Concrete of the same consistency as the groove was next tried, being one part cement, one part sand and three parts crushed granite. This, in some instances, lasted no longer than the blocks, while even where car traffic was infrequent, although the life was much longer the result was not satisfactory. A plate of hard grey cast iron, called by the author, a "frog piece," was at last substituted and drilled and spiked to the ties with the countersunk spikes. The blocks were paved up to this and the whole cemented in.

This cast iron frog piece is, as far as the writer knows, entirely original and has proven a distinct success. Neither the castings or the adjacent blocks have moved in the least, and this method was adopted at seven different turnouts. Three turnouts were left without such provision and blocks, concrete, etc., were tried out, but without success. These will now be fitted with cast iron plates. The pavement, except for the street railway work, has shown up very satisfactory so far. The blocks appear to be uniform and there are no inequalities in the pavement, the only trouble being with the before-mentioned blocks in the track allowance.

The diamond used in the centre of the town is of an altogether too light and flimsy nature. It consists simply of rails four inches in depth, bolted together on plates at the corners and with cross braces in the centre. This should at least have been made of heavier rails of the "girder" type, although there are cast steel diamonds made which have given great satisfaction, and are undoubtedly to be preferred. The result of using the poor diamond above described has been to loosen all the blocks inside and the blocks for a distance of about four feet outside the rails. Repairs are being made on this work at the time of writing.

All headers at street intersections were made of granite concrete, as for curb, and extended twelve inches beyond and to the full depth of the pavement base. This has proven very satisfactory, although the writer believes that it should be carried further—say two feet beyond the pavement, where the intersecting street is unpaved.

The following data may be of interest:—

Price of gravel, deposited on work by teams at contract price	$1.00 cu. yd.
Price of cement deposited in shacks	1.52 barrel
Superficial prices:—	
Excavation and grading	17 sq. yd.
Concrete base (5")	.53 sq. yd.
Blocklaying and ½" cushion	22 sq. yd.
Blocks—teamed on to the work (freight Windsor to Oshawa inclusive)	1.57 sq. yd.
Cubic yard prices for work under track:—	
Excavation	50 cu. yd.
Concrete base	4.68 cu. yd.
Curb and gutter	56 lin. ft.
Average cost of catch basins	24.00 each
Total cost per lineal ft. of 40 ft. road	14.84
" " " " 35 " "	12.98
" " " " 30 " "	11.12

These measurements are taken from face to face of curb.

This last table of total costs includes paving the street, laying curb and gutter, paving the track allowance, extra concrete under the ties, the dismantling and relaying of the track, together with all catch-basins, etc., required.

The total yardage of the pavement was 14,300 sq. yds., and the total cost including the foregoing incidentals was $47,750.00.

GRAND TRUNK EQUIPMENT ORDERS.

Orders have been placed by the Grand Trunk system for locomotives as follows for the coming year: 25 Mikado type from the American Locomotive Works, Schenectady, N.Y.; 50 Pacific type from the Baldwin Locomotive Works, Philadelphia, and 15 large standard switching engines from the Canadian Locomotive Works at Kingston. The Mikado locomotives are larger than any locomotives now in use in Canada. Ten Pacific engines have also been ordered from the Montreal Locomotive **Works.**

RAILWAY FINANCING LAST YEAR

During the past ten years Canada has experienced a wonderful era of expansion in railway building, which has had a marked influence on the development of the country from shore to shore. Probably in no other country have such determined efforts been made to provide transportation facilities, and the rapid rise in importance of Western Canada is an eloquent tribute to this far-sighted and courageous policy. The total railway issues for 1912, according to Mr. E. R. Wood's bond review, were $69,972,320, almost the same as the aggregate for 1910, but falling considerably below the record of 1911—$100,472,700.

Of the railway bond issues, $61,382,320, or 87.73 per cent., found a market in Great Britain, while Canada took .21 per cent., and the United States 12.06 per cent. Great Britain continues to finance the greater part of our railway development. These issues and the markets in which they were sold are set forth in the following table:—

RAILWAY ISSUES.

Company.	Amount.	United States.	Great Britain.
Canadian Pacific Railway 4% Consolidated Debenture Stock	$10,962,320	....	$10,962,320
Grand Trunk Railway 4% Perpetual Consolidated Debenture Stock	12,500,000	....	12,500,000
Grand Trunk Railway Equipment Trust Notes, Series "A"	3,940,000	$3,440,000	500,000
Grand Trunk Railway Equipment Trust Notes, Series "B"	3,360,000	3,000,000	360,000
Canadian Northern Railway 4% Perpetual Consolidated Debenture Stock	6,960,000	....	6,960,000
Canadian Northern Railway 5% Income Charge Convertible Debenture Stock	10,000,000	....	10,000,000
Canadian Northern Railway Equipment Trust Notes, Series "C 1"	2,000,000	2,000,000	....
Canadian Northern Railway Equipment Trust Notes, Series "D 1"	3,000,000	3,000,000	....
Canadian Northern Railway Equipment Trust Notes, Series "E 1"	2,000,000	1,850,000	....
Canadian Northern Pacific Railway 4% Debenture Stock (guaranteed by British Columbia)	5,000,000	....	5,000,000
St. John and Quebec Railway 4% Stock (guaranteed by New Brunswick)	4,250,000	....	4,250,000
Edmonton, Dunvegan and British Columbia Railway 4% Debenture Stock (guaranteed by Alberta)	3,500,000	....	3,500,000
Algoma Central Terminals Limited 5% Bonds (guaranteed by Lake Superior Corporation)	2,500,000	....	2,500,000
	$69,972,320	$8,440,000	$61,382,320

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CONTENTS OF THIS ISSUE.

THE SUGGESTED CHANGES IN THE ONTARIO RAILWAY AND MUNICIPAL BOARD ACT.

The powers of the Ontario Railway and Municipal Board will be considerably enlarged if the proposed Ontario Railway Act, now before the Provincial Legislature, is enacted.

A clause which will have far-reaching effects, is that dealing with the right of the Board to order the construction of spur lines or branches upon the application of an industry. This clause, if adopted, will give the Board power, where any industry lying within six miles of a railway desires shipping facilities and cannot agree with the railway company as to construction and operation of a spur, to order the railway to construct and operate such a line. It may also direct the applicant to deposit in a chartered bank a sum sufficient to construct and complete the spur, and this amount can be paid by the Board to the railway from time to time as the work proceeds.

The aggregate amount so paid by the applicant in the construction and completion of the spur line shall be repaid or refunded to him by the company by way of rebate to be fixed by the Board, out of the tolls charged by the railway company upon the traffic over the branch.

Until the total amount of the cost of construction is repaid, the applicant retains a special lien for this amount on the line; when payment has been received by the applicant, the railway then receives total ownership of the spur.

The old section of the Act, which this clause replaces, entails very roundabout methods, and operates by means of the submission of a by-law. The Board has no mandatory powers under the present Act to enable it to see that its wishes are carried out. This change, if adopted, will be instrumental in securing quicker action, and should work to the advantage of both the railway and the applicant.

Another important change is that providing that a general preliminary plan of a new railway must be filed before the Board. These plans must show the location of the line, the towns through which it passes, the rivers or other railways covered or within a radius of thirty miles. The Board is given the power to approve these plans and to alter them if deemed necessary.

GOOD ROADS.

History repeats itself. The world is again assuming the Roman attitude toward good roads problems. Neither technical education nor country residence is required to appreciate the value of good roads. Any town or village unfortunate enough to be off the railroad map is off every map unless good roads link it up with the world.

Yet it is surprising how many city bred men still believe good roads to be a useless expense—that is, until they buy their automobiles. And even a few farmers can be found here and there who are narrow-minded enough to begrudge a road tax. Educating the public to enthusiasm about good roads has naturally been a slow process, and there have been but few men in Canada who have had the patience and public spirit to keep at the educative task.

Among those whose efforts have not been in vain and who have done a great amount of very praiseworthy work without any hope of personal reward, have been the executive and members of the Ontario Good Roads Association. Special interest is aroused in the annual meeting of this Association at Toronto next week owing to the co-operation which is being accorded to them so liberally at the present time by the Ontario Government. It is understood that the government has practically pledged itself to spend several millions, beginning early in 1914, upon good roads in the more settled parts of the Province. The Ministers are said to look with favor upon the Toronto to Hamilton, Ottawa to Prescott and Toronto to Montreal roads, and the construction of these much-needed highways at a comparatively early date is practically assured.

Had the Ontario Good Roads Association never done any other work, it has fully justified its existence by its splendid promotion and organization work in connection with these three roads. Some of the members of the Association have spent hundreds of dollars personally and given weeks of time to the work without any remuneration whatever. Their main task now seems to be securing Federal co-operation. It is to be hoped that some plan will be brought out at the Convention next week which will result in interesting the Dominion government to a greater extent than the Association has succeeded in doing in the past. With Federal aid the county councils should be able to afford any type of pavement which will be the best and cheapest ultimately, regardless of a possible higher first cost.

CANADA CREOSOTING COMPANY.

Forty acres of land on the river front at Trenton, Ont., have been purchased by the Canada Creosoting Company, which will erect a large plant for creosoting timber of all kinds.

The United States Wood Preserving Company and the American Creosoting Company jointly control the stock of the Canada Creosoting Company, but about one-third of the shareholders of the Canadian company will be residents of Canada, as a block of the stock of the new company is being sold privately in Canada. The head sales office of the new company will be at Montreal, where a tank station will be erected. Creosoting oils will be brought to the Montreal station in tank steamers from Europe. A small tank steamer, specially constructed for going through the St. Lawrence canals, will carry the oil from the Montreal station to the Trenton plant. Mr. E. S. Clements, of the United States Wood Preserving Company will be the Canadian manager, with headquarters at Montreal. President Hurt, of the American Creosoting Co., will be the president of the Canadian company. A. B. Clements, vice-president and general manager of the United States Wood Preserving Company, will be the vice-president.

The company will treat, principally, railroad ties and wood paving blocks, by the vacuum-pressure method. Approximately 150 men will be employed at the start. A plant costing in the neighborhood of $200,000, and with storage space for a million ties, will be erected at once. The company has purchased enough ground to be able to treble the initial capacity of the Trenton plant, and it is thought that within a year the capital of the Canadian company may be greatly increased and the plant considerably enlarged.

The Trenton plant will be practically a duplicate of the United States Wood Preserving Company's Toledo (Ohio) plant, which is said to be the most modern and best-equipped wood preserving plant in the United States. Three cylinders, each 190 feet in length, will be constructed at once for the creosoting process.

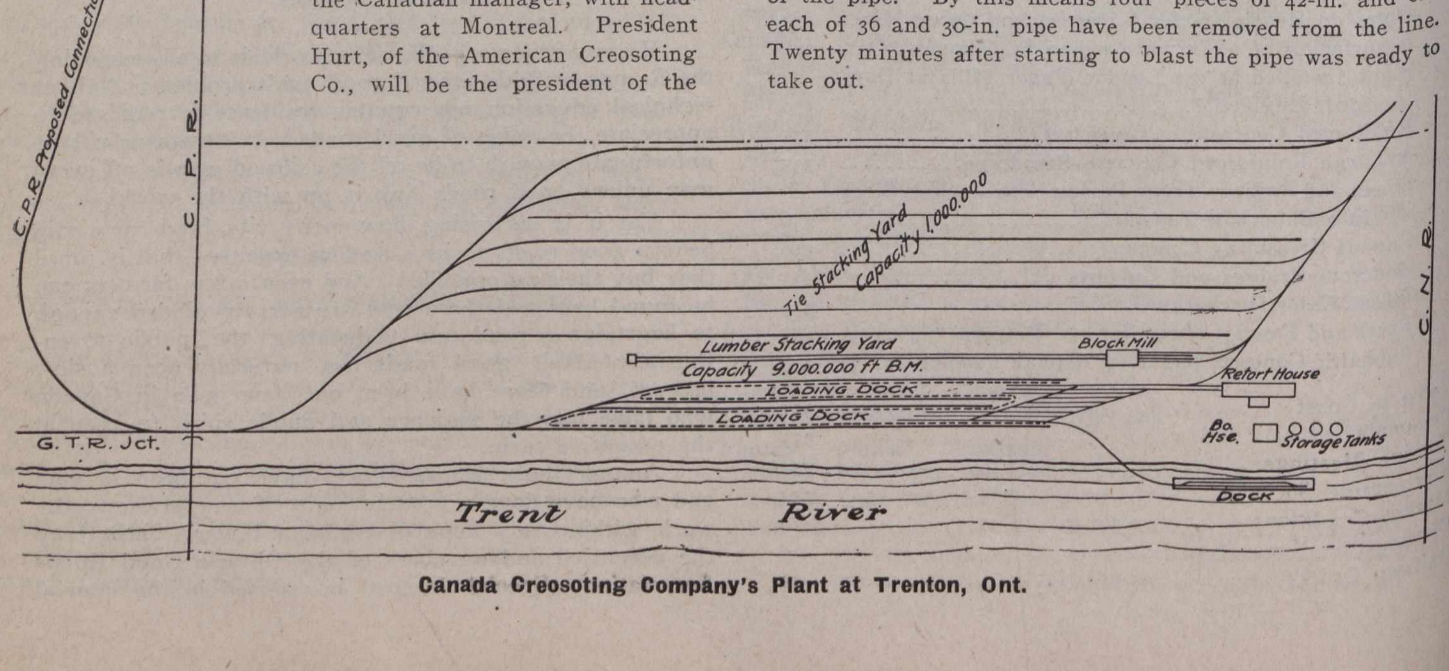
There are already two big plants in Canada engaged in preserving timber—those of the Dominion Tar & Chemical Company, Limited, at Transcona, near Winnipeg, and at Sydney, N.S. The Dominion Tar & Chemical Company produce their own creosoting oils at tar-distilling plants at Sault Ste. Marie, Ontario, and Sydney, N.S. It is not expected, however, that the Canada Creosoting Company will in any way conflict with the business of the Dominion Tar & Chemical Company, as there will undoubtedly be a tremendous call in the next decade in Canada for preservative treatments of timber, and the plants of both companies will probably be worked at full capacity.

The railroads save huge sums by having their ties creosoted, as it adds but little to their cost in comparison to the many years which it adds to the lifetime of the ties. Bridge timbers, piling, sheeting, shop and factory floors and wood for many other purposes can be very profitably creosoted

DYNAMITE FOR REMOVING BROKEN SECTIONS OF CAST IRON PIPE.

At the annual convention of the American Water Works Association, Mr. W. F. Wilcox described the use of dynamite for removing broken sections of large size cast iron pipe.

The method utilized is as follows: Starting in the centre of the pipe to be removed, place ¼ to ⅓ of a stick of 40 to 60 per cent. dynamite on top of the main and cover it with a handful of clay. The blast blows out a small hole or starts a crack. Further shots are shot off working toward each end of the pipe. By this means four pieces of 42-in. and one each of 36 and 30-in. pipe have been removed from the line. Twenty minutes after starting to blast the pipe was ready to take out.



Canada Creosoting Company's Plant at Trenton, Ont.

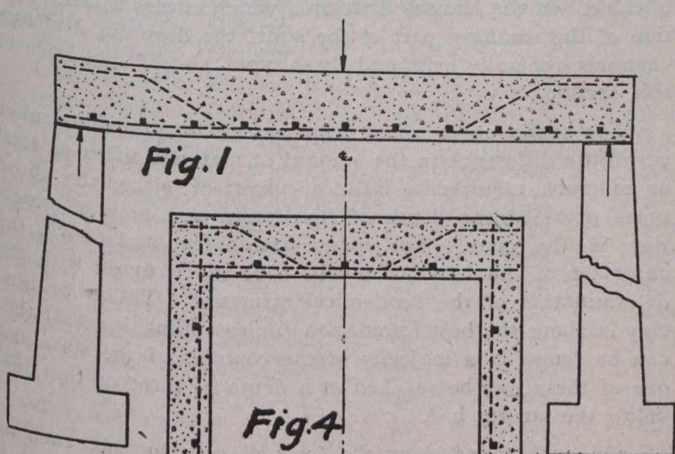
CONCRETE BRIDGES AND CULVERTS*.

In view of the fact that so many of the concrete highway bridges constructed in the last few years are composed of various combinations of simple beams and cantilevers, it may not be out of place before taking up the study of these various types to briefly summarize the laws governing the action of simple beams and cantilevers.

Figure 1 shows a simple slab or beam supported at the ends and loaded as indicated by the arrows. Under the action of the three forces as indicated, namely, the two abutment reactions and the load, the beam tends to deflect or bend at the centre, producing a crack or cracks extending from the bottom of the beam upward. The reinforcement is placed perpendicular to these cracks to carry the tension and thus prevent cracking of the concrete.

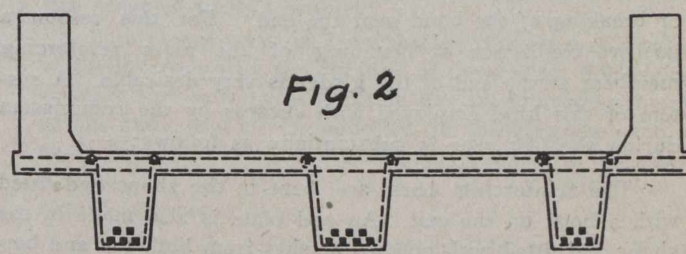
In the preceding case no account was taken of the shearing forces existing in the beam. Without going into the technical details, let it suffice to say that these shearing forces so modify the direct tension that, in addition to the tendency to crack at the centre, there is also a tendency toward the formation of cracks running diagonally upwards and toward the centre between the quarter point and the end of the beam. This is the so-called diagonal tension or shearing crack. Two methods of reinforcing are used to counteract this cracking tendency. In the first one, a portion of the reinforcing members at the bottom of the beam are bent up, running through the concrete at an angle varying from 30° to 45 degrees to the top of the beam and thence horizontally to the end, as shown in Figure 1.

In the second system, vertical stirrups, consisting of a series of vertical rods fastened to the main horizontal reinforcing members, and running up through the beam carry the shear. (This last arrangement is shown in Figure 2, which is a cross sectional view). Since the tendency to crack



is greater towards the ends of the girder than at the centre, the number of stirrups is increased approaching the ends. The method of reinforcing which is used and recommended by the commission consists of a combination of these two methods, using both the bent up rods and stirrups.

Figure No. 3 is a cross section through the body of a reinforced concrete abutment. The darkened portion above the abutment represents the superstructure which rests on the abutment. If an expansion joint between this abutment and the superstructure is constructed to permit free movement between them, the action of the earth pressure has a tendency to produce cracks extending from the back of the abutment toward the face and should be reinforced as shown in this sketch by vertical bars placed near the back face (marked A). Should the expansion joint prove defective, the pressure of the earth under this condition produces a bulging action of the abutment and the necessity for reinforcing steel on the front face is at once apparent.



The various types of concrete highway bridges constructed in this state, adapt themselves to the following classifications:—

Circular and box culverts.
Slab bridges.
Girder bridges and arches.

The girder bridges may be sub-divided into the deck and the through types. In addition, there is the steel truss with concrete floors and abutments.

Taking them up in the order named, Figure No. 4 is of the box culvert type, showing the cross section through the barrel of the culvert. The top, sides and floor of this culvert act as beams under the vertical loads, the horizontal earth pressure, and the upward foundation pressure respectively. Since in each case the load acts from the outside inward, the reinforcing is placed in each case on the inside of the wall or slab. Since there are no expansion joints, no cantilever action can take place and there is no need for reinforcing on the outside of the walls. The top slab is reinforced with bent up bars, as shown, to provide for the diagonal tension as outlined above.

Two types of footings are used in these culverts. The one marked "A," known as a float footing, consists of a heavy floor extending clear across the slab and depending for its stability on the wide distribution of the load. In the type marked "B," the footings directly underneath the walls are carried down to a considerable depth and are connected by a thin floor slab built just strong enough to take the bending due to frost action.

The second type, as the name implies, consists of a slab or beam of concrete (Fig. 1) as wide as is required for the roadway, resting on two abutments. The reinforcing is for simple bending and diagonal tension. Since at each bridge seat there is an expansion joint, the abutments (Fig. 3) are reinforced near both back and front surface with rods running vertically and horizontally. The horizontal rods form a frame-work for the reinforcing, bond the wings to the abutment proper and prevent cracking due to temperature changes in the concrete. The rods in the back face take all the ordinary stress. The front reinforcement is called into action only in case of the failure of the expansion joint to act properly, on this account the vertical reinforcing system in the back is much heavier. Steel bars placed both longitudinally and transversely in the footing, as shown, help to distribute the footing load and to anchor the main reinforcing members.

Fig. 2 is a cross section of a deck girder bridge. The girders are nearly rectangular in section, having a slight

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*From a report by Thos. H. MacDonald and C. B. McCullough, of the Iowa Highway Commission.

draw towards the bottom to admit of easy removal of the forms. These girders, or stems, are thoroughly bonded by means of stirrup rods to the floor slab which rests on top of it, the latter acting both as the floor and as a top flange for the girder. The load is carried by the floor to the girders and by the girders directly to the abutments. The rectangular sections at the side of the roadway serve simply as hand rails and do not help to transmit the load.

Due to the diagonal tension, there is always a tendency for the bent up reinforcing members to fail by a pulling out or breaking of the bond near the end. For this reason a positive connection or fastening of the main reinforcing members at the end of the girder is very desirable. A system of this kind employed with success by the commission during the last year is substantially as follows:—

The reinforcing bars are bent in the shops and fitted with a hook on the end. An end plate is also made in the shops and fitted with notches or slits from both top and bottom of sufficient width to admit the reinforcing bars. These notches are so spaced that when the reinforcing bars are dropped in they occupy the position as computed in the structure. After the bars are dropped in these notches, two small plates about ⅜ of an inch thick are bolted one to each side of the plate both top and bottom, thus completely enclosing and rigidly fastening the reinforcing members.

The other type of a girder (the through, Fig. 5) differs from the type just described, in that the girders, two in number, are placed above and at the side of the roadway. The reinforcing in these girders differs very little in principle from that employed on deck girders. The girders now occupy the space occupied by the hand rail in the deck type and the floor system is hung between the two girders and reinforced transversely in much the same manner as an ordinary slab.

In the last type, the arch, the loading comes upon the structure through the fill above it, produces cross bending in the arch ring and a thrust against the abutments. The type of abutment used commonly is known as the gravity type and consists of heavy mass concrete work depending for its stability on its size and weight. The necessity for immovable abutments will be evident from the following consideration. Any spreading or settling of the abutments will be followed by a downward deflection of the arch ring which in turn produces a tendency to crack at the crown on the lower side and at points near the spring line on the upper side. Any slight settlement of the abutments produces a tremendous stress in the arch ring and the necessity for adequate abutments cannot be too greatly emphasized.

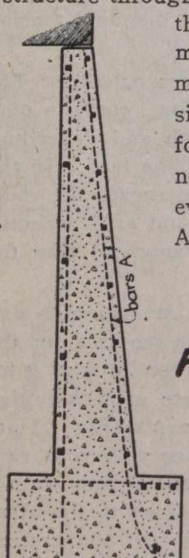

Fig. 3

The circular mold culvert is a combination of the arch and box culvert type and is adapted to small culvert work where an opening not to exceed thirty or thirty-six inches is required.

There is a distinct difference in the shape and size of the drainage lines over this state, those lying within each distinct glacial area being peculiar to that area and the types of waterways built should be selected with reference to the topography. For illustration: In the broad, flat, monotonous stretches of the Wisconsin glacial region which lies in the north central part of the state, the drainage areas are for the most part, flat and not well defined. The channels quickly fill to overflowing and a large percentage of the waterway must be obtained by raising the road grade above the land level.

For such openings, the slab and through girder bridges offer the greatest amount of waterway with the least rise and span. In the construction of bridges in this area, the possibility of the ditch being incorporated in a drainage district must always be guarded against in planning a bridge. An instance is brought to mind of a practically new concrete bridge in Buena Vista county that is to be destroyed because the foundations were not carried sufficiently deep when constructed to provide against the deepening of the channel as part of a drainage district.

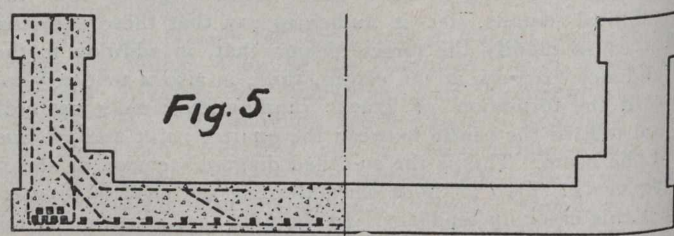

Fig. 5.

In the more rolling areas along the rivers on the east and west boundaries of the state, many of the streams have deep, sharply defined channels. During times of heavy storms the water rushes through the openings in mad little torrents that quickly scour under the foundations unless they are protected by deep footings and floors. For these deeper ravines, the box culvert with ample floors and footings, the slab or deck girder or arch bridge with high rise are all applicable. In the Kansas drift area which covers a large portion of the southern part of the state, the deep cut drainage channels generally exist and these types are all applicable to this section.

For spans up to about sixteen feet it probably makes very little difference in the amount of material either of steel or concrete, required to build a culvert of either the above types provided the depth of foundation and area of waterway is the same. For spans sixteen feet and above, the character of the foundation will have much to do with the determination of the economical structure. Yellow or blue clay is about the best foundation (unless piling is used) that can be found in a majority of the counties of the state and one of these can be reached at a depth of three or four feet below the stream bed.

On such foundations the form of construction, such as the slab or girder bridge, or for the longer spans, steel bridges with concrete floors, seems to be desirable as the expansion joint at each abutment will take care of the expansion and contraction of the structure as well as any settlement of the foundation without causing material cracks.

GREAT WATERWAYS UNION.

A general meeting of the Great Waterways Union will take place at Berlin, Ont., at an early date and representatives of the many municipalities interested in the Welland Canal and St. Lawrence River route development are expected to be present. Henry Holgate, of Montreal, has been advocating on the behalf of the union that the government create a Department of Inland Waterways to study and develop inland navigation.

STORM WATER DISCHARGE.

By R. O. Wynne-Roberts* and T. Brockmann.†

(Continued from last week).

Mr. Symons, in 1878, observed a precipitation in Camden Town, London, England, falling at the rate of 12 inches per hour, continuing for 30 seconds (Moore). In the Central States of America a rate of 3.7 inches for 10 minutes, 2.8 inches for 20 minutes, 2.3 inches for 30 minutes and 1.7 inches for 60 minutes may be expected (Folwell).

The following are a few recorded examples of the intensity of rainfall. Those recorded in Toronto were kindly supplied by Prof. Stupart, the director of the meteorological service.

Year.	Place.	Duration of storm in minutes.	Actual rainfall recorded inches.	Rate of rainfall inches per hour.
1905	Ponders End, England..	19	1.05	3.32
1905	Birmingham, England .	27	1.04	2.31
1905	Birmingham, England .	7½	0.56	4.50
1906	Burnham, England	40	1.73	2.60
1906	Guildford, England	8	0.87	6.68
1907	Worcester, England	8	0.43	3.22
1905	Wallington, England ...	65	2.77	2.55
1869-1891	New York, U.S. ...	10	1.20	7.20
1871-1891	Washington, D.C...	5	0.80	9.60
1905	Washington, D.C.	78	3.11	2.38
1884-1891	Philadelphia, Pa. ..	20	1.60	4.80
	Brooklyn, N.Y.	25	2.60	6.24
1899	Frankfort, Germany	6	0.40	4.00
1899	Frankfort, Germany ...	4	0.267	4.00
1891	Frankfort, Germany ...	6	0.47	4.70
1890	Frankfort, Germany ...	7	0.55	4.70
1903	Toronto, Ontario	5	0.11	1.32
1903	Toronto, Ontario	15	0.60	2.40
1903	Toronto, Ontario	5	0.40	4.80
1904	Toronto, Ontario	7	0.28	2.40
1904	Toronto, Ontario	10	0.51	3.06
1904	Toronto, Ontario	8	0.48	4.80
1905	Toronto, Ontario	5	0.22	2.64
1905	Toronto, Ontario	5	0.25	3.00
1905	Toronto, Ontario	5	0.38	4.56
1907	Toronto, Ontario	5	0.25	3.00
1911	Toronto, Ontario	7	0.54	4.63
1912	Toronto, Ontario	5	0.25	3.00
1912	Almasippi, Manitoba ...	90	0.73	0.50
1912	Crescent Lake, Sask....	30	0.48	0.96
1912	Bardo, Alta.	60	1.20	1.20
1912	Delia, Alta.	45	1.20	1.60
1878	Chicago, Ill.	7	0.97	8.30

It is unfortunate that there is comparatively little information available as to the intensity of rainfall in different parts of various countries. Still, the records suffice to show that the maximum intensity ordinarily lasts only for a few minutes, sometimes at the commencement of a storm, other times in the middle or at the end of it. Furthermore, the area of maximum intensity is usually circumscribed and often located in different parts of a district.

It often occurs that after a prolonged drought, rainfalls of great intensity are experienced. Sometimes it happens that during long periods of wet weather, when the surface is already saturated and the sewers are flowing practically full, a rainfall of greater intensity will fall and thus give rise to floods—unless the sewers are of more than ordinary capacity.

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† Civil Engineer, Regina.

If records of rainfall in wet districts are carefully studied, it will doubtless be found that the intensity is relatively lower there in comparison with drier districts.

Figure 1 shows two curves illustrating rainfall intensity. Curve I. represents moderate amounts of rainfall observed at Edgebaston, Birmingham, England. This curve is a copy of one prepared by Mr. Wallington Butt, who stated that it is more or less applicable to other parts of England, and by it storm water discharges can be estimated in that country. Curve II. represents the mean highest rainfalls observed in Germany, and this generally constitutes the maximum intensity assumed in that country.

Both curves serve to show that the intensity of a rainfall varies inversely with the duration of the storm. For example, a rainfall of 20 minutes duration has an equivalent intensity of one cubic foot per second-acre in Birmingham, and 1.73 cubic feet per second-acre in Germany, whilst a ten minutes rainfall has an intensity of 1.33 and 2.36 cubic feet per second-acre respectively.

Although the following diagram (Fig. 1) will be found useful as a graphical method of ascertaining the factor of rainfall intensity with sufficient accuracy, it will doubtless be acknowledged that it is highly desirable to ascertain the intensity in any particular district by means of reliable automatic recording rain gauges, of which the simplest is usually the best.

In countries where a considerable accumulation of snow takes place, such as in Canada, and this melts in spring time, it is a matter of great importance to give this point careful thought, for it often occurs that rain falls when the temperature rises, and this causes serious floods. No statistics are at present available on this feature of our enquiry, so it must be left for the reader to bear it in mind when undertaking investigations.

In addition to ascertaining the rainfall intensity in any district, it is practically certain that it will not be of uniform intensity throughout that area, especially if the area is at all extensive. An interesting example of this is referred to by Sir Maurice Fitzmaurice, who, when discussing Mr. Lloyd Davies' paper, remarked that very heavy showers were extremely local in character, and mentioned a case where rain falling heavily at Hampstead caused a sewer at King's Cross, London, to be gorged and some lives to be lost, although, where the men were working no rain had fallen. The distance between the two places is only about three miles.

Careful observations have been made, in Germany, for instance, and it was found that the mean intensity of rainfall became less as the drainage area increased in size, or the sewers increased in length, beyond a certain radius from the point of heavy rainfall.

It may be stated that observations were made in Breslau, Germany, on an area of about 1,700 acres to ascertain the relative intensity at three stations located as far apart as possible, and it was found that during a period of 760 minutes rain fell simultaneously for only 90 minutes at two stations, and at the three stations for only 28 minutes. In view of these facts, it was not justifiable to assume a uniform intensity over a large area. Further investigations were, therefore, instituted so as to ascertain in what manner the maximum intensity "qr" varied, with the result that it was found that the intensity diminished to "qr/2" in a distance of about 10,000 feet from the point of observation.

It may be objected that it is too unfavorable to base a conclusion on such limited number of instances where rain fell on all three or two stations as the same time, and to ignore the larger number of rainfalls which occurred at solitary stations.

It is submitted, however, that such precaution is justified by the limited extent of the observations, by their local character, and by the fact that storms are not stationary, but travel in different directions, sometimes across a district, sometimes diagonally and at other times along the major axis of the drainage area, in the latter case a proportionately larger quantity of storm water will naturally have been precipitated and have to be dealt with.

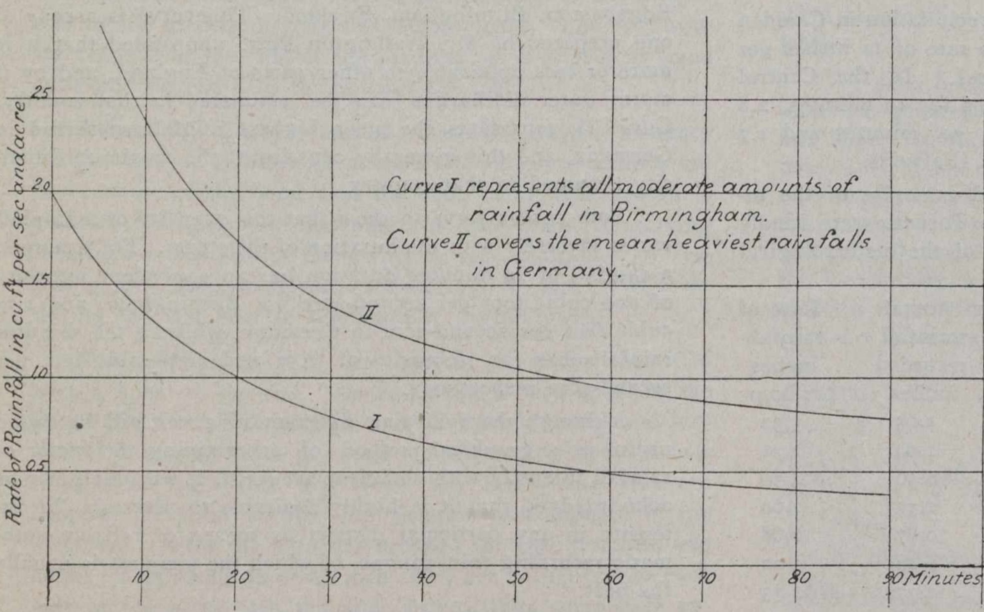

Fig. 1.—Rainfall Intensity Curves.

It may also be contended that it would be more accurate to take, instead of the length of 10,000 feet, a value depending directly on the intensity, for as the area decreases, the intensity increases. The results of the observations, however, are not sufficient to ascertain, with any degree of accuracy, the relation which subsists between the intensity of a rainfall and the area and, moreover, those downpours taken into consideration have some features in common in the manner of occurrence, and do not differ very much in their intensity.

For the sake of simplicity, it may be assumed that the curve representing the decrease from the maximum intensity "q_r," to that at the remote part of a district, "$q/2$," is that of a parabola, so that the intensity and extent of any storm are represented by a solid of revolution, generated by the revolution of a parabola along its axis, and a section along this axis will therefore be, as shown in Figure 2, at M, A, N, O, P, being limited on the top by two parabolas. The ratio between the solid of revolution and a cylinder representing a uniform rainfall, then, will give the coefficient "I" by which the highest intensity assumed has to be multiplied so as to obtain the average intensity of storm, and his factor is called the "coefficient of intensity."

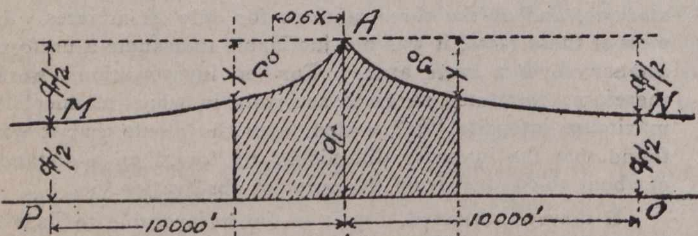

Fig. 2.

Let "A" be the zero point, then the parameter "p" is found from $10{,}000 \times p = \left(\frac{q}{2}\right)^2$ or "p" equals $\frac{q^2}{40{,}000}$; the equation of the parabola will be

"y^2" $= \frac{q^2 \times x}{40{,}000}$.

The circumference described by the centre of gravity in revolving is $2 \times 0.6\,x \times \overline{x}$, therefore the content of the solid of revolution corresponding to the shaded area (cylinder minus parabola) is

$x^2 \pi q - \frac{2}{3} x\, y\, 1.2\, x\, \pi = x^2 \pi (q_r - 0.8\, y)$

and "I" $= \frac{x^2 \pi (q_r - 0.8\, y)}{x^2 \pi q_r} = 1 - 0.8 \frac{y}{q_r}$.

As $\frac{y}{q_r} = \sqrt{\frac{x}{40{,}000}}$, "I" $= 1 - \frac{0.8\sqrt{x}}{\sqrt{40{,}000}}$.

The downpour will yield the maximum storm if "A" is located in the centre of the drainage area, and then "x" may be assumed to be equal to one-half of the length of the sewer and it will be:—

$$I = 1 - .0028\sqrt{L}$$

"L" is the length of the sewer in feet.

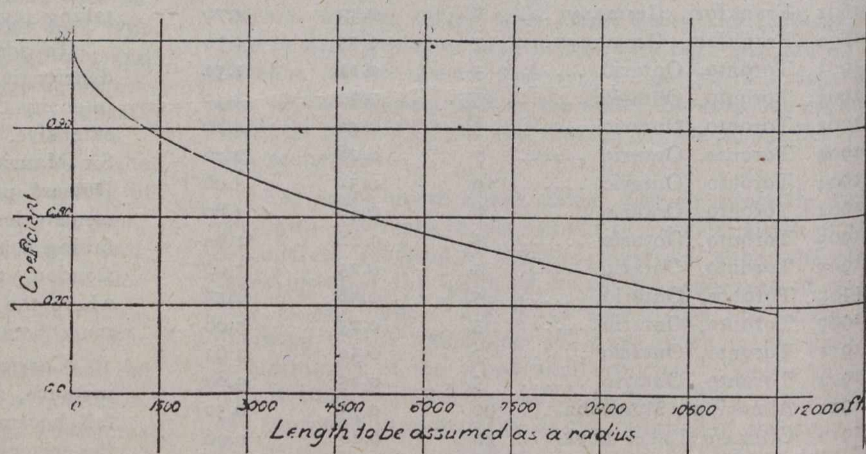

Fig. 3.—Diagram for Determining the Coefficient of Intensity.

If "q_r" be taken equal to "y" then from the equation of the parabola, the values "x" equals 40,000 feet and "L" equal to $2x$ equal to 80,000 feet will be obtained. This, therefore, is the limit of the application of the above equation and obviously this limit will accordingly not be attained even in the case of very extended sewerage system.

The following table will give the coefficients of intensity ascertained by means of the foregoing formula $I = 1 - 0.002\sqrt{L}$:

L in feet.	Coefficiency of intensity.	L in feet.	Coefficiency of intensity.
500	.937	5,000	.802
750	.923	5,500	.793
1,100	.9115	6,000	.782
1,250	.900	7,000	.766
1,500	.892	7,500	.758
1,750	.883	8,000	.750
2,000	.875	9,000	.733

L in feet.	Coefficiency of intensity.	L in feet.	Coefficiency of intensity.
2,500	.860	10,000	.720
3,000	.847	12,000	.692
3,500	.835	15,000	.660
4,000	.823	20,000	.605
4,500	.813		

Figure 3 has been prepared to give the above information.

As an explanation of the foregoing table and diagram, it may be stated that if the calculations in connection with a proposed sewerage system is based upon a high rainfall intensity of, say, 1.428 cubic feet per second-acre, the sewer being, say, 1,650 feet long, the coefficient for which in the above table and diagram is 0.89. Then 1.428 × 0.89 equals 1.271 cubic feet per second-acre, which is the intensity to be provided for when designing the sewers.

It will be observed that it is necessary in this connection to reduce the rainfall, which is measured in inches depth, to cubic feet per second-acre. As one inch of rain per hour on one acre is practically equal to one cubic foot per second-acre, it will be a simple matter to make the conversion. Thus

1 inch of rain per hour equals 1 cu. ft. per second-acre
0.5 inch of rain per hour equals 0.5 cu. ft. per second-acre
0.1 inch of rain per hour equals 0.1 cu. ft. per second-acre
and so on.

(To be continued).

JANUARY PRECIPITATION.

Precipitation was deficient from Eastern Saskatchewan to the Highlands of Ontario, throughout the Maritime Provinces, with the exception of the vicinity of Halifax, and also over the southern part of Vancouver Island. Elsewhere the amount recorded exceeded the average. The snowfall in British Columbia was phenomenal, especially in the Coast districts. At Vancouver nearly five feet of snow fell during the month and sleighing was general for three weeks.

A considerable quantity of snow fell in British Columbia during the month, and at its close sleighing was good in most localities, the depth in southern districts being about eight inches and in the north, five feet. In Southern Alberta the snow covering slightly exceeded one inch, but elsewhere in the Western Provinces the depth was generally more than six inches, and reached twenty-five in northern parts of Manitoba. Much less snow than usual was on the ground from Ontario eastwards, there being about thirty inches in northern districts which decreased as southern localities were approached, where in Ontario and the Maritime Provinces the ground was bare of snow.

The following table, included in the report of the Meteorological Office, Toronto, shows the total precipitation of fourteen stations for January, 1913:

	Depth in inches.	Departure from the average of twenty years
Calgary, Alta.	1.32	+0.86
Edmonton, Alta.	2.40	+1.69
Swift Current, Sask.	0.50	—0.16
Winnipeg, Man.	0.80	—0.17
Port Stanley, Ont.	5.70	+2.37
Toronto, Ont.	4.36	+1.55
Ottawa, Ont.	4.10	+1.11
Kingston, Ont.	4.90	+2.03
Montreal, Que.	5.10	+1.35
Quebec, Que.	3.70	+0.50
Chatham, N.B.	3.40	—0.18
Halifax, N.S.	6.30	+0.45
Victoria, B.C.	4.50	—0.04
Kamloops, B.C.	3.10	+2.15

FACTS AND FANCIES ABOUT SEWAGE DISPOSAL.*

By Gilbert Thomson, M.A., F.R.S.E., M.Inst.C.E.

The number of methods of dealing with sewage has been enormous. They may be broadly grouped into four divisions:—

No treatment.
Land treatment.
Chemical treatment.
Bacterial treatment.

These are not necessarily distinct, as more than one may occur in the one installation. But the four divisions are convenient.

The no treatment system is of very limited application. It has been ousted in succession from small streams, from large streams, and from tidal rivers. It has a rather precarious hold even on the sea, except where a specially suitable place of discharge can be found. In our limited time we may with advantage confine our consideration to those methods of sewage disposal which involve more or less of treatment.

I might begin by referring to one or two popular fancies. One is that filtered sewage is good drinking water. It is not. A sewage effluent may be very good as a sewage effluent, and very bad as drinking water. Good drinking water should be free from any trace of sewage contamination; a good effluent is one from which the polluting matter has been partly removed, and in which the remainder has been rendered harmless. The analytical results which indicate the most satisfactory sewage effluent would utterly condemn the water for drinking purposes. There are many good effluents, many indifferent ones, and I fear still more bad ones, but I would decline the invitation to drink even the best.

Another popular fancy is that there is a "best" system of sewage disposal. It is quite natural that an inventor or patentee should think that his particular system is the best. We give him credit for perfect good faith even when he tries to convince us that it is equally applicable to works draining a populous area of many square miles, requiring inlet sewers in which men walk upright, where there is an equipment of elaborate machinery, where the tanks have to be in a more or less populous neighborhood and of great size, and where the discharge is into a large tidal river like the Clyde—the illustrations are from the Glasgow works—and to a work of modest dimensions, with no machinery, discharging into an inland stream. We take the liberty of thinking, however, that his opinion is prejudiced.

Even when a man is not an inventor, if his experience has been limited to a few examples, he is very likely to draw rash conclusions, and to assume that because he has seen one system doing better than another the relative result will always be the same. In doing so he forgets two things—first, the suitability for local conditions; and, second, the personal element.

In speaking of the personal element, I do not refer chiefly to those who are responsible for the design and supervision of the construction, nor to our useful, if somewhat rough, friends who actually carry it out; but I refer especially to the men who, day in and day out, with sometimes little recognition of their care and ability, keep the works in proper order. I should like to repeat what I have often said before, that the efficiency of any sewage work, whether it is a big one like Glasgow or Birmingham (from both of which we have illustrations), or a work too small to require the

* Abstract of lecture before the Sanitary Association of Scotland, September 18th, 1912.

whole attention of one man, depends quite as much on its management as on its construction.

One of the most interesting duties in connection with sewage is to be called in, as a doctor is called in, because the works are "sick." Without touching on the highly controversial subject of medical men and contract practice, I am strongly of opinion that something of this nature might be useful in sewage works. The big work has its own skilled man, but the little work has to call in the doctor when its digestion goes out of order, and, like the human patient, it is apt to be pretty bad before the doctor, in the shape of the engineer, is called in. Much unpleasantness might be saved by a regular visit, and the administration of the necessary tonic before the patient's condition became serious.

It might be of interest to mention some of the common ailments:—

Overwork is the most common of all. This may either be general overwork, due to the deficient size of the installation; or local overwork, due to the bad distribution of the work, in other words, of the sewage. This last is very common.

Faulty tanks, allowing too much suspended matter to reach the filter.

Filters clogged and requiring to be re-made.

Improper drainage of filters.

Some mechanism out of order.

These points will be better appreciated after some consideration of the various methods. The remedy, of course, depends on the exact nature of the trouble.

Of all the methods of treating sewage, it is probably correct to say that none has surpassed land treatment in producing a good effluent. It is not many years since (in England) it was compulsory to finish off with land, whatever the preliminary treatment may have been; and the general abandonment of land in recent years has not been because of its inefficiency, but on account of other difficulties, chiefly that of getting enough land. No further back than last month I heard at the York Congress of the Royal Sanitary Institute a strong plea for land treatment, and, although some of the arguments in its favor were far from convincing, it must not be supposed that this treatment is altogether out of date. The stock argument, that it is a more "natural" process, reminded me of the reasoning of Mause Headrigg in "Old Mortality." She objected to her son being called on to use fanners for winnowing corn, "instead of waiting patiently for whatever dispensation of wind Providence was pleased to send."

Natural or not, land treatment may give excellent results; but when one bears in mind that under the system of broad irrigation, from which alone good farming results are to be expected, an acre will not serve for a population of more than 100, and that with the most concentrated system of land filtration an acre of actual filter is needed for every thousand of the population, it is easy to see why processes which do the work on a very much smaller area are usually preferred. It is simply out of the question in the case of a large town to obtain a sufficient area of suitable land; even a small town finds it difficult. An artificial filter will deal with a population of many thousands per acre.

Almost every form of treatment begins with rough screening, which removes such things as dead cats and old boots, and which in a large work results in a great volume of refuse. This is usually combined with a settling tank for the rougher grit. In Glasgow this is taken out of the tank by a dredger.

In land treatment there is no other preliminary, but the chemical and bacterial methods of treatment have preliminaries which are vital parts of the process.

Before speaking of chemical and bacterial methods, it is desirable to refer to the composition of sewage from a mechanical point of view.

Sewage consists of water, with polluting material partly dissolved and partly suspended.

The scum is of little consequence. Most of it ultimately disintegrates and either dissolves or sinks. The sludge is a very different matter.

It has certain manurial value, but with about 10 per cent. of manure you have about 90 per cent. of water. It does not pay, therefore, to carry it far, and in the immediate neighborhood of a large town the demand for manure is limited. It is possible to get rid of some of the water by allowing the sludge to settle in large tanks, as is done in the Glasgow works at Dalmuir and Shieldhall. Still more can be got out by drainage on filters, as is well illustrated by a series of views from Birmingham. A still further advance is made by filter-pressing, as is done at Glasgow Dalmarnock works. And, finally, at the same works a valuable and compact manure is produced by further drying the filter-pressed sludge. But raw sludge is of so little value that where water transport is readily available, as at Dalmuir and Shieldhall, the method which is adopted is that of taking the sludge out to sea and depositing it in deep water.

Leaving for a time the question of filtration, let us look at the bacterial process. Their war-cry for a time was "No sludge," but that, I think, has been dropped or modified.

Everybody has heard of the septic tank. When I had the honor of addressing this congress from the chair in 1898, I described a pilgrimage which I had made a year or two earlier to Exeter to see what was then the last word in sewage purification, and which was hailed as a complete solution of the sewage problem. It has certainly been of great value, but the septic tank is in itself no solution of the sewage problem. The Royal Commission has reported adversely on some of its chief claims.

The name took the public fancy. It became familiar to many who knew little about sewage, and the microbes who inhabit these tanks have become familiar friends. Many people seem to be on fairly intimate terms with them, and to know that their chief occupation is that of eating each other up. I have been gravely assured that this process of mutual consumption can be watched, and I have heard an emphatic condemnation of a tank on the ground that no microbes could be seen in it. Thousands of people who know nothing whatever about sewage purification will tell you, without the slightest hesitation, that the proper way to purify sewage is by means of a septic tank. They sometimes spell it "sceptic," which describes exactly the proper mental attitude to such a proposal. It is no exaggeration to say that nowadays one of the difficulties in dealing with sewage is to convince those concerned that anything more is needed than to construct a big cesspool, and to call it a septic tank.

It is quite true that the early septic tanks wrought for years with no great increase in the material deposited in the bottom, and it was not unnatural to conclude that most of the solids were dissolved. But experience has shown that when accumulation of sludge does not occur in the tanks, it simply means that the fine suspended matter is passing away in the tank liquor. Unfortunately, many tanks have been constructed with no convenient arrangement for removing sludge, in the belief that no sludge would be produced. This is one of the fancies for which local authorities have had to pay dearly. What happens is that, after a certain amount of accumulation has taken place, the liquid leaves the tank heavily charged with matters in suspension.

So far we have dealt with what may be called the early history of sewage disposal. This brings us to a comparatively new field of inquiry. The liquid which leaves the tanks, and which is now usually termed "tank liquor," contains always a large proportion of matter in solution, and usually a quantity of matter in suspension. With the former it is quite easy to deal. We can't remove it, but by proper appliances it can be oxidized and rendered harmless.

The required method is filtration. If the sewage is passed through a proper filter, to which the oxygen of the air has free access either periodically or all the time, the filter, like the tank, will soon swarm with minute life. This life is instrumental in bringing the sewage matters and the oxygen into intimate contact, so that the sewage matters are burnt up by the oxygen.

In order to have this done effectively, it is essential that the sewage be divided into the smallest possible parts, so that the microbes may come into contact with all of it. This division may be effected in various ways—the liquor, for example, may be sprinkled on the top of a filter, or it may be poured on in bulk and allowed to find its way through the interstices of the material—but somewhere and somehow it has to get through small openings. If the liquor contains only dissolved matter, it will pass freely through these small openings; if it contains suspended matter, it will soon choke them up.

The importance of this is not realized as it ought to be. It is good policy to spend money and trouble in getting a tank liquor which is reasonably free from suspended matter.

It is much cheaper to clean a tank than to clean a filter, even when the tank has not been well designed for the purpose; while with a well-designed tank there is no comparison. This is a branch of sewage engineering to which far too little attention has been given.

I do not mean that it has been ignored by those who are really experts; but in ordinary works the design of the tanks is usually the weakest part. Even in works of considerable size, the tanks too often show evidences of the belief that the sludge will disappear, and that no provision for its removal is required.

On the other hand, some very ingenious systems have been devised, with the object of getting the solid matters out of the main stream of sewage. There are three tanks of which a good deal is heard—Dr. Travis's hydrolytic tank, the Imhoff tank, and the Dortmund tank. Of the two former I will only say that in them the sludge which settles out from the sewage is encouraged to pass through openings in the bottom of the channel into a part of the tank where there is little or no flow. With the long and comparatively narrow tank common a few years ago, the sludge might settle down, but it could not get out of the current. Everybody is familiar with the manner in which a person may be swept along by a crowd in a narrow passage; he may be most unwilling to go on, but he has no choice. But if there are side passages or recesses here and there, the person who wishes to stay behind gets into one of these. Much the same principle applies in these tanks. They are somewhat complicated in construction, and the question of whether or not it is worth while to use them must be settled for each individual case.

The Dortmund tank is better known. It was first employed at the town of Dortmund, whence its name. For a while it had no great popularity, but it is now very often used. In principle it is little more than a deep pit with sloping sides, down which the sewage matters fall, while the liquid passes over the top.

But supposing the sludge to be satisfactorily caught in such a pit, how is it to be got out? It would be a very troublesome operation, and a very offensive one to empty the tank every now and then to get out the sludge. If you make a hole in the side of a tank full of sewage the sewage will run out, or if the hole is made into the part which is filled with sludge, it is the sludge that will come. If you provide a pipe dipping down into the sludge, and coming out below the surface level, the sludge will come out near the surface level; and if you provide the pipe with a valve, it is quite easy to wait until the sludge has gathered, open the valve, and let the sludge run out, and shut the valve whenever the sludge has been removed. By such an arrangement it is easy to remove the sludge as often as may be necessary without disturbing the tank to any extent, and therefore with comparatively little smell. It is well to say comparatively, for there are few operations connected with sewage that don't smell more or less, and it is of the utmost importance to have as little handling of it as possible. Various methods of dealing with the sludge after it has been got out have already been mentioned. Another method which has recently been devised is to spread in a special tank a layer of stable litter, then run in a layer of sludge, spread another layer of stable litter, and so on. The great point is to get the sludge as free from water as possible, so that it may become fairly portable, and thus available as manure. The liquid which drains from it is, of course, very foul, and unless it can be properly filtered in some other way it is pumped back into the tank among the sewage. Fortunately, there is not very much of it.

There is no time to do more than mention "slate beds," where the sewage enters not a large unoccupied space, but a space provided with numerous slate shelves, so that the deposited matters may be more fully disintegrated.

We may now return to the liquid which leaves the tank, whether the tank is a precipitation tank, a septic tank, or a sedimentation tank. It is no longer crude sewage, but what is known as "tank liquor." It is very far from being pure water, and has still to be filtered.

Here our friends the microbes have their second innings, and, with all respect to the anaerobic branch of their family, most sanitarians are agreed that the aerobic microbes are more to be trusted. We have got out of the sewage everything that we can by a rough process of settlement, aided by any breaking up or dissolving that may take place. The tank liquor is tolerably clear—if our tanks have done what they should—but it is full of putrescible matter, dissolved if not suspended. We want to get this putrescible matter oxidized, and so rendered harmless.

We do this by getting it into close contact with pieces of filtering material, swarming with bacteria, and see that these bacteria are liberally supplied with oxygen, that is to say, with fresh air. This close contact is not so easily got.

Two methods have been adopted. The one is what is known as the "contact bed," the other as the "percolating" or "streaming" or "continuous" filter.

The contact bed consists of a big tank, usually about 3 or 4 feet deep. It is filled with the "filtering medium"—usually clinker or some such material broken into pieces about the size of road metal. Many materials have been used, and many different sizes have been tried.

Of course, there is still a good deal of space left between these pieces, and into this space the sewage is run. Roughly speaking, the filtering material occupies about two-thirds of the total space, and the liquid about one-third. By this means of working there is no question that the sewage must be brought into close contact with the filtering material Suppose the filtering material to be covered with a multitude of microbes, each fed up with fresh air and hungry for sewage, it follows that when the sewage is let into the bed these microbes promptly set to work. After a while, however, they become fed up with sewage, and want more fresh air, not to speak of time for digestion. The liquid, therefore, must be run out of the bed to let the fresh air in. The whole process is an intermittent one, and consists of four stages—filling, standing full, emptying, standing empty. It follows that there must be more than one bed, so that at all times one may be filling. Very often the number is considerable; five is a convenient number, so that one may be in each of the four stages, and one taking a rest.

The original septic tank installation was of this type. A most ingenious set of tilting buckets was employed, so that all the changes were wrought automatically. When a bed was full the flow of sewage was turned on to the next. When it had stood full for a certain time it was allowed to empty, all by these automatic contrivances. Since then many other methods have been devised for controlling the operations, most of them depending on syphon action.

This method of filtering sewage has fallen on evil days. The Royal Commission has issued a report which is not very favorable to it, and in the future it is probable that contact beds will be the exception. The report showed that, when original cost and running expenses are both taken into account, contact beds are not such an economical method of dealing with sewage as are percolating filters. It must not be forgotten, however, that they have certain advantages, not the least being that they are much less likely to cause offensive smells than some other methods. But any method of sewage disposal is ready to cause smell on very slight provocation.

The other method is to sprinkle the tank liquor on the top of the filter like a shower of rain, or like a dose from a watering can. At first sight it might appear that nothing is easier than to water the surface of a filter with fair uniformity, giving each part an equal share of the liquid. In fact, it is far from easy, and of the failures which have occurred in sewage purification a big proportion may be traced to failure in this particular.

Look at the matter in this light—In designing a sewage filter, the first thing to be considered is the amount of work it has to do. Having ascertained this as nearly as he can, with, of course, a proper provision for uncertainty and for future extension, the engineer calculates what size of filter will be needed, and the filter is constructed of a capacity to fit the work to be done. A well-designed system is based on each part of the filter doing its proper share of the work. But if the distribution is so bad that one part of the filter does no work or far too little, it follows that the other parts get more than their share to do. Some filters have been designed rather extravagantly, and are too big for their work, in which case bad distribution may not cause any visible harm. But if the filter has been well proportioned, bad distribution soon results in bad effluent and offensive smells. A sewage filter which is working well has no serious smell; but a filter which has been overwrought, and has become "sewage sick," takes a prominent place among nuisance producers. It is very false economy to have bad distribution; it must be paid for either in extra size of filter or in nuisance.

In big works, where power-driven appliances are in use, it is quite common to use power to drive the distributors. In that case it is fairly easy to get good distribution, and, besides, in such a place there is skilled attendance to put right at once anything that may go out of order. There is little doubt but that the best results which have been attained have been under these conditions.

But in small works the sewage has to distribute itself, the power being derived from its own fall. More than that, the fall that can be given is usually very limited, and many inventors have been at work on the problem of how to distribute the liquid which comes from the tank, keeping in view that the surface level of the filter should be as nearly as possible the same as the level of the sewage in the tanks. It often happens that between the sewage level in the tanks and the level of the bottom of the filters there is nothing to spare, and any fall required to distribute the liquid must simply mean a reduced depth of filter.

Much ingenuity has been displayed in meeting this requirement, and a number of prominent methods will be illustrated. They may first be classified as follows:—

1. Distributors which are fixed.
2. Distributors which move.

The first class contains the following:—

(a) A gridiron of perforated pipes.—This is not often used on a large scale, being apt to choke up, and also being irregular in its distribution.

(b) Trays.—These have the advantage of not requiring much fall, but it is difficult to maintain even distribution. The large surface of exposed liquor adds materially to the risk of nuisance from smell, while the trays hinder the free access of light and air. Installations of this type are very common.

(c) Sprays.—These are only available when there is plenty of fall. In favorable circumstances they are the cheapest of satisfactory distributors.

They have been adopted as the standard type at the Birmingham filters, probably the largest in the kingdom. In Scotland the Crieff sewage is distributed by these sprays with very satisfactory results. Many varieties of jet have been tried, the object being to secure freedom from choking, and at the same time to throw the spray horizontally over the bed and not to send it much into the air. Otherwise the risk of smell is considerable. The jet shown was a great improvement on the earlier types.

Of the second, the following are in common use:—

(a) Revolving distributors with perforated arms.—The perforations are on one side of each arm, and the reaction causes the whole apparatus to revolve. The system is a clever adaptation of "Barker's Mill," which was otherwise little more than a scientific toy. The trouble is that the holes are necessarily small, and thus apt to choke. Various mechanical devices have been applied to the clearing of the holes, and several very ingenious systems of suspending the apparatus so as to revolve with the minimum of friction are on the market. The principle is in very common use.

(b) Revolving distributors driven by central turbine.—In order to avoid the difficulty of the small holes this method of driving has been used. The reaction of the arms is not needed for driving, and so the arms are open troughs, over the edge of which the liquor spills as they revolve.

There is a risk in either of these types that with a very small flow the revolution may stop. It is usual, therefore, to pond up the tank liquor when the flow is small, so that it is discharged not in a continuous dribble, but in a succession of rushes. These distributors, therefore, in dry weather are usually working and resting in fairly quick succession. There is nothing against this, except that the necessary syphons introduce an extra complication.

(c) Revolving distributors of the water wheel type.—These are driven by the weight of the liquor, which is delivered into the buckets by a feed pipe. The distributor is a long drum, pivoted at one end and carried by a driving wheel on a rail at the other. As the drum turns it moves along and so spills the liquor over the filter. It is a more cumbrous and costly apparatus, but does excellent work. On a large scale care has to be taken to balance it by having two arms, otherwise the wind might hinder its movement.

All the foregoing require that the filters should be circular. It sometimes happens that the ground is more suitable for rectangular filters, or that percolating filters are used to replace contact beds, hence the introduction of the next type.

(d) Travelling distributors of the water wheel type.—In principle these are identical with those just mentioned. The feed, however, instead of coming from the centre, is taken by a syphon from a long trough. In the most recent type the weight is carried chiefly by one central rail, a small proportion resting on the trough for balancing purposes.

Most of the methods are the subject of patents, some of them of many different patents. Each manufacturer, quite naturally and not improperly, does his best to keep his wares before the public, and the wares, as a rule, are good. The variety of apparatus, all designed to perform the same apparently simple operation, is somewhat bewildering. When a man sees an installation working well, and producing a high-class effluent, he is naturally inclined to say that the system must be a first-class one, and he is apt to conclude that he need only adopt that particular system in order to produce as good a result elsewhere. That is a dangerous fancy. It is not only the effluent that must be considered. I have in my mind two works, one producing an effluent which to appearance is spring water, and which gives analytical results which are entirely satisfactory. The other produces an effluent which shows evident signs of being other than clean water. One might say that the first works were much better than the other. But I believe that the latter works are purifying the liquid to a greater proportional extent than the former. The sewage in the one case is from a residential town, with a liberal supply of water; the sewage in the other is from a mining village, where the supply of water is very limited. When the comparison is made between the sewage entering the works and the effluent in each case, it is realized that effluent alone is not a fair test.

It is not only the distribution that affects the efficiency. The essence of filtration is that the liquid and the oxygen of the air are brought into contact. It is useless to supply the filter with liquid unless it is also supplied with air, and if the bottom of the filter gets choked either with water or with rubbish, there is no chance of air getting into the body of the filter. Accordingly, it is important to have the bottom of the filter freely supplied with channels by which the water can escape and the air enter.

Look also at the different kinds of filter wall. In some cases the filtering material is merely a bing with no containing walls; in others there is a "dry-stone dyke" to hold in the material, or there is a rough wall built of the bigger lumps of the material itself. Then, again, there is ordinary brickwork, and in other cases there is "pigeon-hole" brickwork. The idea of the open walls is to allow air to enter more freely, but the general opinion now is that if the bottom of the filter is well ventilated it matters little how the walls are constructed. The real test is economy in its broad sense, and it is rather amusing, on the one hand, to hear it argued that walls are an unnecessary extravagance, and that the material should be allowed to take its own natural slope; and on the other, that the extra material thus required and the extra land occupied would cost more than the wall. The relative cost, of course, depends largely on local conditions, but personally I would be inclined to have a wall of some sort, however rough, if it were only for the sake of keeping the place more tidy.

The subject of sewage disposal is becoming more and more complicated. At one time the choice of system was made, as it were, in the mass; the decision to adopt land treatment, chemical treatment, or septic tank treatment, covering practically all the general arrangements. Nowadays the usual thing is to adopt what may be called septic treatment, but which I much prefer to call "bacterial" treatment, as being a wider description; but the choice then is only beginning. The shape and capacity of the tanks, the method of working them, the removal and subsequent disposal of sludge, the application of the liquor to the filters, the shape of the filters, their depth, the kind of material and the size of the pieces, the drainage of the filters—these are only some of the problems which have to be solved, and the solution of one does not always help in the solution of the next. Frequently the balance between different courses is extremely fine, and in many instances it would be impossible to say dogmatically that one was right and the other wrong.

There are quite a number of different systems, any of which will work quite efficiently if the details are skilfully designed and the working efficiently supervised, and it is quite possible that two equally competent men may arrive at quite different designs. The economical solution of the problem calls for two things—a thorough knowledge and clear grasp of general principles, and a thorough and detailed investigation of the special conditions of the place. The most economical system is not always that which is apparently cheapest; on the contrary, economy and cheapness are often very different things.

The difficulty is emphasized by the fact that the money to be spent is public money, and those responsible for the expenditure are very properly jealous of any unnecessary cost. Not being themselves experts, they are apt to take any promise of cheapness at its face value, forgetting that an estimate may be very reliable or very sanguine. In England, where many local authorities invited competitive schemes, and adopted what promised to be the cheapest, many cases have occurred where the final cost bore only a distant resemblance to the original estimate. The intervention of the local government board, who called pointed attention to these costly object lessons, pretty well killed that practice there, and I have sufficient faith in Scottish shrewdness to expect that the lesson will be taken here without many such experiences.

EXPANSION IN RAILWAY BUILDING.

The Canadian Northern Railway and its subsidiary companies disposed of $28,960,000 securities in 1912, or 41 per cent. of the year's total.

The Grand Trunk Railway required heavy financing during 1909, 1910 and 1911, but in 1912 issued no securities to the public. Under a special arrangement, we understand, certain advances were made to this company by the Dominion Government. The Grand Trunk Railway in 1912 issued securities aggregating $19,800,000, as compared with $10,000,000 in 1911. The issuance of equipment trust notes by the Grand Trunk Railway is worthy of special mention in that for the first time this company availed itself of this modern and reasonable method of financing its equipment requirements. These obligations found a ready market in the United States.

The Canadian Pacific Railway made its annual issue of 4 per cent. consolidated debenture stock, aggregating $10,962,320, proceeds of which were used to build branch lines, purchase additional steamships and to acquire the bonds of subsidiary companies. Its larger requirements are met by the sale of capital stock, of which $18,000,000 was issued in 1912, with an additional $60,000,000 in immediate prospect.

The construction of the Edmonton, Dunvegan and British Columbia Railway is the first serious attempt to open up that part of Northern Alberta commonly known as the Peace River country.

The aggregate production of lode gold in British Columbia up till 1912 was 3,421,000 ounces (fine) of $70,497,000. The placer gold amounted to $72,139,000, making a total of $142,636,000. The Yukon shipped out $169,000,000.

ASPHALTIC CONCRETE AND SHEET ASPHALT PAVEMENTS.*

Asphaltic Concrete Pavement.

Subgrade and Drainage—H-1.—The subgrade and drainage shall be constructed to the plans, profiles and specifications of the City Engineer.

Foundation—H-2.—The foundation upon which the pavement is to be laid shall be concrete, and shall be prepared according to specifications and requirements of the City Engineer.

Wearing Surface.—H-3.—Upon the prepared foundation shall be laid the asphaltic concrete wearing surface, consisting of a mixture of selected, hard, crushed stone, gravel and sand, mixed with asphaltic cement as hereinafter specified. The wearing surface shall be of a thickness of 2 in. after thorough compression with a roller weighing not less than 200 lbs. to the inch width of tread, and shall contain between 7 and 12 per cent. bitumen soluble in carbon disulphide, depending on the mineral aggregate treated.

Mineral Aggregate—H-4.—Stone.—The stone shall be crusher run varying in size from a maximum of ½ in. to the smallest particles retained on the finest mesh screen commonly used on crushing plants. The dust or fine screenings should be completely removed from the stone, as it is usually excessive and irregular in quantity and necessitates the use of a greater amount of asphaltic cement. Sufficient sand and filler, varying from 8 to 15 per cent., shall be added to completely close and fill the interstices of the mineral aggregate. After being rolled the surface shall present a granular appearance, showing that the structural body of the pavement is crushed stone. The asphaltic cement shall be in sufficient quantities to bind and fill the mixture as specified above, but not to flush to the surface as free cement under the roller.

Sand.—Sand shall be clean, moderately sharp and well graded, and shall be free from loam.

Mix.—The mix shall consist of a mixture of sand and stone as above specified, so as to comply with the following specifications:—

Passing 100 x 200-mesh sieve	8 to 15%	
Passing 80-mesh sieve	10 to 20%	Mineral aggregate
Passing 40-mesh sieve	15 to 30%	
Passing 10-mesh sieve	30 to 40%	
Passing ¼-in. ring	15 to 40%	
Passing ½-in. ring	0 to 30%	

Method of Mixing—H-5.—The aggregate shall be thoroughly dried in properly designed driers before mixing with the bitumen. The driers shall be of the revolving type, thoroughly agitating and turning the materials during the process of drying. When the aggregate is thoroughly dried and heated to a temperature of from 200 to 350° F. (depending upon the asphaltic cement used), it shall, immediately before cooling or exposure to moisture, be mixed with the hot asphaltic cement as hereinafter specified.

The asphaltic cement shall be melted in a tank arranged so that the heat can be properly and easily regulated. When melted and raised to a temperature of from 200 to 350° F. (depending upon the asphaltic cement used), it shall be combined in proper proportions with the hot aggregate and immediately mixed in a properly designed mixer with revolving blades until a thorough and intimate mixture of the ingredients has been accomplished, and the particles composing the aggregate evenly and thoroughly coated with the asphaltic cement.

* Specifications adopted by Vancouver, B.C.

Laying—H-6.—Paving mixture at a temperature of from 200 to 350° F. shall be hauled on to the street in dump wagons, and shall be kept well covered with canvas to retain the heat, and dumped and shoveled into place. While it is still hot it shall be evenly spread with hot iron rakes, and while still pliable shall be rolled with a steam roller as above specified, so that when ultimate compression is accompilshed the surface shall be even and true to grade. The rolling must be steadily kept up, lengthwise, crosswise and diagonally, and continued until all roller marks disappear and surface shall give no appearance of further compressibility. Along the curb, around manholes and catch basins, where roller cannot reach, the required compression shall be made by the use of hot iron tampers. Tamping shall be done as quickly as possible after the material is spread, while it is still hot and pliable.

Joints—H-7.—All contact surfaces along curb, around manholes, castings, etc., shall be painted with an asphaltic cement before the paving mixture is laid.

The paving shall be done continuously, so that the number of joints between the hot and cold material shall be reduced to the minimum. When it is not practicable to lay it continuously, and a joint is unavoidable, the edge of the cold material shall be trimmed down to a rough feather edge and the surface where the joint is to be made painted over with asphaltic cement, and the hot material raked over the feather edge and thoroughly rolled; or, instead of trimming the cold material, joint strips may be used, consisting of strips of canvas about 18 in. wide, with three parallel lines of ¾-in. ropes sewed on the underside about 3 in. apart; the joint strips shall be laid on the feather edge of the freshly raked materials, with the upper rope at the line where the thickness begins to decrease, and the rolling completed on top of the canvas as for finished pavement.

Asphaltic Cement—H-8.—The asphaltic cement shall be composed of refined asphalt and flux.

Refined Asphalt—H-9.—The refined asphalt to be used under these specifications shall be approved and in every way satisfactory to the City Engineer. The refined asphalt shall be equal in quality to the recognized standards, and must comply with the following requirements:—

(1) All shipments of any one kind of material shall be uniform in consistency and composition, and shall not vary more than 15 points in penetration at 77° F.

(2) When the refined asphalt is made into an asphaltic cement by use of flux hereinafter specified it must produce an asphaltic cement of the required penetration satisfactory to the condition of the street, and at the penetration at which it is to be used it must comply with all requirements set forth herewith for asphaltic cement.

Fluxes—H-10.—The fluxes shall be residues obtained from the distillation of paraffin, asphaltic or semi-asphaltic petroleum. They must be homogeneous and of a uniform gravity, and free from all signs of cracking, and must comply with the following requirements:—

(1) They shall have a specific gravity at 77° F. of between 0.92 and 1.04.

(2) When 20 grms. of flux are maintained at a temperature of 170° C. for five hours in a cylindrical vessel 2½ in. in diameter there must not be volatilized more than 5 per cent. by weight.

(3) They shall not flash below 180° C. when tested in a New York State closed oil tester.

(4) They shall be soluble in carbon bisulphide to the extent of 39.5 per cent.

The asphaltic cement prepared as above described must comply with the following requirements:—

(1) It must be homogeneous, adhesive, viscous and must not be affected by the action of water.

(2) It shall have a penetration of between 50 and 70 per cent. at 77° F.

(3) When 20 grms. of asphalt are maintained at a temperature of 150° C. for five hours in a cylindrical vessel 2½ in. in diameter there must not be volatilized more than 5 per cent. by weight, nor shall the original penetration be reduced thereby over 50 per cent.

(4) The asphaltic cement shall not be so susceptible to changes in temperature as to have a penetration varying more than 125 between 0 and 45° C., and it shall have a ductility of not less than 5 cms. at 25° C., and not less than 2 cms. at 0° C. (Dow Standard).

Squeegee Coat—H-11.—Immediately after rolling, and while the pavement is still warm, a thin coat of pure bituminous cement shall be spread over the surface by means of rubber squeegees, and upon this shall be spread a thin layer of stone chips or other suitable material, dry and free from dust, and containing no particles which shall be less than ¼-in. ring nor greater than ½-in. ring. After applying dressing the surface shall be again rolled until it presents a smooth and finished appearance, subject to the approval of the Engineer.

Sheet Asphalt Pavement.

Subgrade and Drainage.—[The Subgrade and Drainage and the Foundation Requirements are the same as those given above for asphaltic concrete.—Ed.]

Binder Course—C-3.—The binder course shall consist and be made up and laid of what is known to the asphalt trade as "compact," or "coles" binder, and shall be composed of hard, clean broken stone (no gravel will be allowed or used in the mixture), which shall pass an opening 1 in. in diameter, the voids in which are filled with finer stone passing an opening ⅜-in. in diameter, while the voids in the mixed stone shall be filled with a well-graded sand.

To the stone and sand aggregate (after thorough mixing) sufficient asphalt cement shall be added to thoroughly coat the mineral aggregate with bitumen without showing any excess on compression with a hot tamper.

The stone and the asphalt cement shall be heated separately to such a temperature as will give, after mixing, a binder of the proper temperature for the material employed. The stone when used must be at a temperature between 200 and 335° F. The asphaltic cement when used must be at a temperature between 250 and 350°. The asphaltic cement and stone shall be thoroughly mixed by machinery in such proportions that the resulting binder shall have life and gloss without an excess of asphaltic cement, and the mixing shall be continued until a homogeneous mixture is produced in which all the particles are thoroughly coated with asphaltic cement.

Asphaltic Cement for Binder — C-4.— The asphaltic cement for the binder course shall have a consistency of at least 20 points, as indicated by the Bowen machine, higher than that in use in the surface.

Laying — C-5.—The binder mixture, prepared in the manner described, shall be brought to the street in wagons at a temperature between 200 and 325° F., and shall be covered with canvas covers while in transit. The temperature of the binder mixture within these limits shall be regulated according to the temperature of the atmosphere and the working of the binder. On reaching the street it shall at once be dumped on the concrete, and then be deposited roughly in place by means of hot shovels, after which it shall be uniformly spread by means of hot iron rakes, and then at once be thoroughly compacted by tamping or rolling. The depth of the finished binder shall average not less than 1 in. in thickness, and its upper surface shall be parallel to the surface of the pavement to be laid. The surface, after compression, shall show at no place an excess of asphalt cement, and any spot covering an area of 3 sq. ft. or more showing an excess of asphalt cement shall be cut out and replaced with other material. All binder which shows lack of bond, or that is in any way defective, or which may become broken up before it is covered with wearing surface, must be taken up and removed from the street and replaced by good material properly laid in accordance with these specifications at the expense of the contractor. Binder when laid shall be followed and covered with wearing surface as soon as is practicable in order to effect the most thorough bond between the binder and the wearing course. The binder course shall be kept as clean and free from traffic as is possible under working conditions. If necessary it must be swept off immediately before laying the wearing surface on it.

No binder shall be laid when in the opinion of the Engineer the weather conditions are unsuitable, or unless the concrete on which it is to be laid is dry and has set a sufficient length of time.

Wearing Surface.—C-6.—The wearing surface shall be composed of sand, filler and asphaltic cement, as hereinafter specified, mixed in the proper proportions.

Sand.—The sand must be clean, moderately sharp, and having a grading not varying more than 5 per cent. from the following limits; i.e., 5 per cent. from the three combined gradings:—

Standard Grading.

Bitumen		12.0%	
Passing 200-mesh	14.0%	.0000%	
Passing 100-mesh	12.6%	0.1702%	
Passing 80-mesh	10.4%	0.1405%	= 31.7%
Passing 50-mesh	23.0%	0.3108%	
Passing 40-mesh	10.0%	0.1351%	= 44.6%
Passing 30-mesh	9.0%	0.1210%	
Passing 20-mesh	5.0%	0.0620%	
Passing 10-mesh	4.0%	0.0540%	= 23.7%

Filler.—The filler must be thoroughly dried limestone, or other inorganic dust, or Portland cement. The whole of it shall pass a 30-mesh screen, and at least 70 per cent. of it shall pass a 200-mesh screen. From 5 to 15 per cent. of dust shall be added to the surface mixture, depending upon the kind of dust used, grading of sand, and traffic conditions of the street.

Asphaltic Cement.—[Same as for asphaltic concrete, given above.—Ed.]

Preparation.—C-8.—The wearing surface shall be composed of sand, filler and asphalt cement of the character elsewhere specified, and mixed in proper proportions. The sand and asphaltic cement shall be heated separately to such a temperature as will give, after mixing, a surface mixture of the proper temperature for the materials employed. The sand when used must be at a temperature between 250 and 375° F. The asphalt cement when used must be at a temperature of between 250 and 350° F. The filler shall be added to the hot sand in the required proportions and the two thoroughly mixed. The asphalt cement in the proper proportions shall then be added, and the mixing continued for at least one minute in a suitable apparatus until a homogeneous mixture is produced in which all particles are thoroughly coated with asphalt cement. The weights of all materials entering into the composition of the wearing surface shall be varied in the presence of inspectors as often as may be required, and the Engineer or his representative shall have access to all parts of the plant at any time.

Laying—C-9.—The surface mixture, prepared in the manner above described, shall be brought to the street in wagons at a temperature between 230 and 350° F., and shall be covered with canvas covers while in transit. The temperature of the surface mixture within these limits shall be regulated according to the temperature of the atmosphere and the working of the mixture and the character of the materials employed. On reaching the street it shall at once be dumped on a spot outside of the space on which it is to be spread. It shall then be deposited roughly in place by means of hot shovels, after which it shall be uniformly spread by means of hot iron rakes in such a manner that, after having received its final compression by rolling, the finished pavement shall conform to the established grade and have a thickness of not less than......in. Before the surface mixture is placed all contact surfaces of curbs, manholes, etc., must be well painted with hot asphalt cement. After raking, the surface mixture shall at once be compressed by rolling or tamping, after which a small amount of cement shall be swept over it, and it shall then be thoroughly compressed by a steam roller weighing not less than 200 lbs. to the inch width of tread, and the rolling being continued until a compression is obtained which is satisfactory to the Engineer. Such portions of the completed pavement as are defective in finish, compression or composition, or that do not comply in all respects with the requirements of these specifications, shall be taken up, removed and replaced with suitable material, properly laid in accordance with these specifications, at the expense of the contractor. Whenever so ordered by the Engineer a space of 12 in. next the curb shall be coated with hot asphalt cement, which shall be ironed into the pavement with hot smoothing irons.

No wearing surface shall be laid when, in the opinion of the Engineer, the weather conditions are unsuitable, or unless the binder on which it is to be placed is dry. The finished pavement must be well protected from all traffic by suitable barricades until it is in a proper condition for use.

Requirements—C-10.—The finished pavement shall contain not less than 12 per cent. of bitumen soluble in cold carbon disulphide, depending upon the mesh composition and the character of the sand used and the traffic to which it is to be subjected, but in all cases sufficient asphalt cement must be used to properly coat all the particles of the mineral aggregate. It must also contain not less than 10 per cent. of mineral matter passing a 200-mesh sieve, and not less than a combined total of 25 per cent. passing the 200-, 100- and 80-mesh sieves. On streets of light traffic, when the Engineer has approved the use of a coarser sand or mixture than that specified for general use, the surface mixture must contain not less than 5 per cent. of mineral matter passing a 200-mesh sieve, and not less than a combined total of 18 per cent. passing the 200-, 100- and 80-mesh sieves. The maximum amount of 200-, 100- and 80-mesh material in the pavement will be regulated according to the kind of sand and asphalt used and the traffic upon the street on which the pavement is to be laid, subject to the maximum requirements elsewhere herein specified under sand and filler.

The above limits as to mesh compositions and per cent. of bitumen are intended to provide for such permissible variations as may be rendered necessary by the raw materials used and the character of the work to be done. The composition of the wearing surface may be varied within the limits above specified at the discretion of the Engineer, depending upon the kind of sand, filler and asphalt used and traffic conditions upon the street or streets to be paved.

Condition at Expiration of Guarantee—C-11.—In addition to the proper maintenance of the pavement during the period of guarantee the contractor shall, at his own expense, just before the expiration of the guarantee period, make such repairs as may be necessary to produce a pavement which shall:—

(a) Have a contour free from depreciation of any kind exceeding ½ in. in depth, as measured between any two points 4 ft. apart on a line conforming substantially to the original contour of the street.

(b) Be free from cracks, showing disintegration of the surface mixture.

(c) Contain no disintegrated surface mixture.

(d) Not have been reduced in thickness more than ⅜ in.

(e) Have a foundation free from such cracks or defects as will cause disintegration or settling of the pavement or impair its usefulness as a roadway.

GOOD ROADS EXHIBITION.

The first Good Roads Exhibition to be held in Canada will be in the Dairy Building, Toronto Exhibition Grounds, from the 24th inst., to March 1st.

The dates were set to be concurrent with the Toronto Motor Show and with the annual meeting of the Ontario Good Roads Association, which will be from the 26th to the 28th inst. The Exhibition will be under the management of Messrs. Hartley Robinson and E. M. Wilcox. The manager's office is 62 Temperance Street, Toronto.

Single fare on all railways for the entire week has been secured, and a good attendance is anticipated, both at the Motor Show and the Good Roads Show. There will be a number of interesting exhibits by the Patterson Manufacturing Company, Rocmac Road Construction Company, Ontario Bridge Company, Wettlaufer Brothers, The Canadian Engineer, Canada Cement Company, the University of Toronto, Sawyer-Massey Company, etc.

Delegates will be present at the meeting of the Ontario Good Roads Association from most of the cities, towns, villages and counties throughout the province, and highway officials from various parts of Canada and the United States will be present to address the meetings. Some of the speakers will be Sir Lomer Gouin, the Premier of Quebec, who has aroused that province this year to the importance of good roads; Colonel Sawyer, chairman of the Massachusetts Highway Board; Honorable A. R. Reaume, Minister of Public Works for the Province of Ontario; W. A. McLean, provincial engineer of highways for Ontario. County road organizations, construction of roads, maintenance, federal aid, provincial aid, and other subjects will be discussed.

The members of the association will undoubtedly find the Good Roads Show, which will be very handy to their meetings, both interesting and attractive. Both the practical and theoretical sides of road building will thus be presented, as sections of roads with practical demonstration will be seen at the show.

The Ontario Good Roads Association is a very important body of public-spirited men who have accomplished, among other things, the abolition of statute labor in many townships in Ontario; the adoption of the Highway Improvement Act in twenty counties, in which 3,771 miles of highway have been assumed for improvement; the expenditure of $3,393,507 in the improvement of county roads, one-third of which was paid by the province; and the appropriation by the province in 1912 of an additional $1,000,000 for the purposes of the Highway Improvement Act.

CONVENTION OF CANADIAN ELECTRICAL ASSOCIATION.

The Executive Committee of the Canadian Electrical Association of Toronto have decided to hold the next annual meeting of the association at Fort William on June 23, 24, 25. About 600 delegates and members from every part of the Dominion will be asked to attend.

COAST TO COAST.

Halifax, N.S.—The work of constructing the new No. 2 terminal pier at Deep Water has commenced. The construction, which is in charge of the Nova Scotia Construction Company is very interesting. At present concrete piles of 11 to 23 tons in weight are being driven into the sea bottom by a 16-ton steel hammer; 1,818 of these piles are necessary for the foundation of the pier.

Detroit, Mich.—The Great Lakes Engineering Works will construct three great steel car ferries for the Grand Rapids and Northwestern Railway to be used on Lake Michigan between Ludington and Manitowoe. The probable cost will be in the neighborhood of $1,500,000, and the boats will be built on the same style as the SS. Astabula, of the C.P.R. Company, but with a length of 350 feet.

Edmonton, Alta.—The provincial telephone system yielded a net profit of $62,283 for 1912. During the last six years the net profits have been $407,582. Premier Sifton reported that $2,000,000 will be spent to increase and extend the telephone system until every farmer and resident is accommodated with this utility, which is almost as necessary as railroads.

Montreal, Que.—Mr. Jos. Irving, in company with Hon. Clifford Sifton, is in London enlisting the support of English capital in the formation of the International Cement Company, of Hull, which was temporarily abandoned last spring. The company proposes to use a new process in the manufacture of cement and to have a capitalization of $10,000,000.

Montreal, Que.—The Canadian Ice Company, which is incorporated under a federal charter for $500,000, is doing its utmost to obviate the threatened ice shortage. Additional storage buildings are being built.

TORONTO UNIVERSITY ENGINEERING SOCIETY ANNUAL DINNER.

Thursday evening, February 13th, witnessed the twenty-fourth annual dinner of the Engineering Society of the University of Toronto, held at McConkey's, King Street West, Toronto. The affair was a general success, the attendance altogether being nearly three hundred members and guests. The programme of the evening had been arranged with care, and all present seemed to appreciate and enjoy it. Mr. J. E. Ritchie, the president of the Society, had the chair. The speakers of the evening were: Sir Edmund Walker, chairman of the Board of Governors of the University; Dr. R. A. Falconer, president of the University; Dean Galbraith; Dr. W. H. Ellis; Mr. David Molitor, consulting engineer and designing engineer of locks and dams of the Panama Canal, three years professor of Civil Engineering at Cornell University; Messrs. J. B. and J. W. Tyrrell; Mr. J. S. McCannell, president and general manager of the Milton Pressed Brick Company; Mr. J. L. Morris, the oldest graduate of the "School," and Messrs. T. V. McCarthy, D. A. Mutch and F. C. Mechin. Excellent music was supplied throughout the evening by the Science orchestra and the Science octette and others. The menu cards, of admirable and original design, were furnished by the Eugene Dietzgen Company.

Class '09 of the Faculty of Applied Science of the University of Toronto held a re-union dinner at the St. Charles in Toronto, Friday evening, February 14th. Over sixty members of the class were present on this occasion. Mr. W. D. Black, the toast master, was unanimously elected president for the ensuing three years.

PERSONAL.

RAY R. KNIGHT, recently appointed city engineer of Fort William, has left for that city.

JAMES G. LINDSAY, engineer and waterworks manager of Belleville since April, 1908, has resigned his position.

MR. JOS. D. EVANS has resigned his position as chief engineer of the Montreal Tramways Company to become construction manager of the Electric Bond and Share Company, of New York City.

MARCIL PEQUEGAT, honor graduate of '08 and instructor of drawing of the School of Practical Science, Toronto, has been appointed city engineer of Berlin, Ont. Forty applications were received for the position.

MR. ALVIN SCHLARBAUM, B.A.Sc., has severed his connection with Messrs. Smith, Kerry and Chace, as assistant engineer on the Healey Falls development, to accept the position of hydro-electric engineer for the Riordon Pulp and Paper Company, Limited, of Hawkesbury and Merritton, Ont.

H. T. HAZEN, government engineer in charge of the Hudson Bay Railway terminal at Port Nelson, has arrived in Winnipeg after completing the 700-mile journey from the mouth of the Nelson River. On Mr. Hazen's report will depend the Minister of Railways' decision on the location of the Hudson Bay port.

MONSIEUR J. M. F. de PULLIGNY, ingenieur en chef des Ponts et Chaussees, et Directeur, Mission Francaise d'Ingenieurs aux Etats-Unis, New York City, on February 11th delivered an illustrated lecture on "The Public Service of Roads in France," before the graduate students in Highway Engineering at Columbia University.

MR. ALFRED STILL has resigned his position as chief electrical engineer to the mines department of the Algoma Steel Corporation of Sault Ste. Marie, Ontario, Canada, to take charge of the courses in electrical design at the School of Electrical Engineering, Purdue University, LaFayette, Indiana. Mr. Still, who is a member of both the British and American Institutes of Electrical Engineers, has made a special study of hydro-electric developments and long-distance transmission of electric energy. He has lately returned from a trip to Denver, Salt Lake City, and San Francisco, where he has visited many of the important power systems.

OBITUARY.

MR. WM. JOHNSTON SPROULE, M.E., died at his home at St. Lambert, Que., February 6. The deceased was a member of the Canadian Society of Civil Engineers and was connected with the Montreal habor commissioners for 30 years as assistant engineer, retiring two years ago.

GEORGE WILLIAM MAYNARD, mining engineer, who introduced the Thomas basic steel process into the United States, and who had been widely known in the West and abroad as a consulting engineer, died at Boston, Mass., on February 13. His home was in New York, where he was born in 1839. He was one of the original members of the American Institute of Mining Engineers.

JOHN FRITZ, one of the best-known mechanical engineers in the United States, died at the age of ninety years at his home in Bethlehem, Pa., on the 13th inst. Among other positions of responsibility held at various times by Mr. Fritz were the general superintendency of the Cambria Iron Works, at Johnstown, Pa., and the general superintendency of the Bethlehem Steel Works. In 1864 Mr. Fritz built a rolling mill at Chattanooga, Tenn., for the United States Government, and in the years that followed he was an active factor in the development of the Bessemer process, the acid and basic open-hearth and the electric furnace. He was a past president of the American Society of Mechanical Engineers; a past president of the American Institute of Mining Engineers; a member of the American Society of Civil Engineers; an associate fellow of the American Academy of Arts and Sciences; and honorary vice-president of the Iron and Steel Institute of Great Britain. From the last named society he received the Bessemer gold medal in 1893 for notable service in advancing the manufacture of steel. He was awarded the John Fritz medal by the United Engineering Societies in 1902, and the Elliott Cresson medal in 1910.

AMERICAN SOCIETY MECHANICAL ENGINEERS.

The American Society of Mechanical Engineers will hold a joint meeting with the Verein Deutscher Ingenieure in Germany, beginning June 22nd, 1913. The principal industries of Germany will be visited, including the Krupp Iron Works, at Essen.

COMING MEETINGS.

ILLUMINATING ENGINEERING SOCIETY.—A joint meeting of Societies will be held at the Republican House, Milwaukee, Wis. Feb. 22, 1913.

THE CLAY PRODUCTS EXPOSITION.—To be held in the Coliseum, Chicago, Feb. 26th to Mar. 8th.

ILLINOIS WATER SUPPLY ASSOCIATION.—The Fifth Annual Meeting of the Association will be held at the University of Illinois, Campaign-Urbana, Ill., March 11th and 12th, 1913. Secretary, Edward Bartow.

NATIONAL PAVING BRICK MANUFACTURERS' ASSOCIATION.—Annual Meeting will be held March, 3, 4 and 5, 1913, in the Green Room, Congress Hotel and Annex, Chicago, Ill. Secretary, Will P. Blair.

CANADIAN MINING INSTITUTE.—Annual Meeting will be held at Chateau Laurier, Ottawa, March 5th, 6th and 7th. H. Mortimer Lamb, Windsor Hotel, Montreal, Secretary.

CANADIAN ELECTRICAL ASSOCIATION.—Annual Convention will be held in Fort William, June 23, 24 and 25, Secretary, T. S. Young, 220 King Street W., Toronto.

THE INTERNATIONAL ROADS CONGRESS.—The Third International Roads Congress will be held in London, England, in June, 1913. Secretary, W. Rees Jeffreys, Queen Anne's Chambers, Broadway, Westminster, London, S.W.

THE INTERNATIONAL GEOLOGICAL CONGRESS.—Twelfth Annual Meeting to be held in Canada during the summer of 1913. Secretary, W. S. Lecky, Victoria Memorial Museum, Ottawa

ENGINEERING SOCIETIES.

CANADIAN SOCIETY OF CIVIL ENGINEERS.—413 Dorchester Street West, Montreal. President, Phelps Johnson; Secretary, Professor C. H. McLeod.

KINGSTON BRANCH—Chairman, A. K. Kirkpatrick; Secretary, L. W Gill; Headquarters: School of Mines, Kingston.

OTTAWA BRANCH—177 Sparks St. Ottawa. Chairman, R. F. Uniacke, Ottawa; Secretary, H. Victor Brayley, N.T. Ry., Cory Bldg. Meetings at which papers are read, 1st and 3rd Wednesdays of fall and winter months; on other Wednesday nights in month there are informal or business meetings.

QUEBEC BRANCH—Chairman, W. D. Baillairge; Secretary, A. Amos; meetings held twice a month at room 40, City Hall.

TORONTO BRANCH—96 King Street West, Toronto. Chairman, E. A. James: Secretary-Treasurer, A. Garrow. Meets last Thursday of the month at Engineers' Club.

VANCOUVER BRANCH—Chairman, G. E. G. Conway; Secretary-Treasurer, F. Pardo Wilson, Address: 422 Pacific Building, Vancouver. B.C.

VICTORIA BRANCH—Chairman, F. C. Gamble; Secretary, R. W. MacIntyre: Address P.O. Box 1290.

WINNIPEG BRANCH—Chairman, J. A. Hesketh; Secretary, E. E. Brydone-Jack; Meets every first and third Friday of each month, October to April, in University of Manitoba, Winnipeg.

MUNICIPAL ASSOCIATIONS

ONTARIO MUNICIPAL ASSOCIATION—President, Mayor Lees, Hamilton. Secretary-Treasurer, Mr. K. W. McKay, County Clerk, St. Thomas, Ontario.

SASKATCHEWAN ASSOCIATION OF RURAL MUNICIPALITIES.—President, George Thompson, Indian Head, Sask.; Secy-Treasurer, E. Hingley, Radisson, Sask.

THE ALBERTA L. I. D. ASSOCIATION.—President, Wm. Mason, Bon Accord, Alta. Secy-Treasurer, James McNicol, Blackfalds, Alta.

THE UNION OF CANADIAN MUNICIPALITIES.—President, Chase. Hopewell, Mayor of Ottawa; Hon. Secretary-Treasurer, W. D. Lighthall, K.C. Ex-Mayor of Westmount.

THE UNION OF NEW BRUNSWICK MUNICIPALITIES.—President, Councillor Siddall, Port Elgin; Hon. Secretary-Treasurer. J. W. McCready, City Clerk, Fredericton.

UNION OF NOVA SCOTIA MUNICIPALITIES.—President, Mr. A. S. MacMillan, Warden, Antigonish, N.S.; Secretary, A. Roberts, Bridgewater, N.S.

UNION OF SASKATCHEWAN MUNICIPALITIES.—President, Mayor Bee, Lemberg; Secy-Treasurer, W. F. Heal, Moose Jaw.

UNION OF BRITISH COLUMBIA MUNICIPALITIES.—President, Mayor Planta, Nanaimo, B.C.; Hon. Secretary-Treasurer, Mr. H. Bose, Surrey Centre, B.C.

UNION OF ALBERTA MUNICIPALITIES.—President, F. P. Layton, Mayor of Camrose; Secretary-Treasurer, G. J. Kinnaird, Edmonton, Alta.

UNION OF MANITOBA MUNICIPALITIES.—President, Reeve Forke, Pipestone, Man.; Secy-Treasurer, Reeve Cardale, Oak River, Man.

CANADIAN TECHNICAL SOCIETIES

ALBERTA ASSOCIATION OF ARCHITECTS.—President, R. W. Lines, Edmonton; Hon. Secretary, W. D. Cromarty, Edmonton, Alta.

ASSOCIATION OF SASKATCHEWAN LAND SURVEYORS.—President, J. L. R. Parsons, Regina; Secretary-Treasurer, M. B. Weeks, Regina.

ASTRONOMICAL SOCIETY OF SASKATCHEWAN.—President, N. Mc Murchy; Secretary, Mr. McClung, Regina.

BRITISH COLUMBIA LAND SURVEYORS' ASSOCIATION.—President, W. S. Drewry, Nelson, B.C.; Secretary-Treasurer, S. A. Roberts, Victoria, B.C.

BRITISH COLUMBIA SOCIETY OF ARCHITECTS.—President, Hoult Horton; Secretary, John Wilson, Victoria, B.C.

BUILDERS' CANADIAN NATIONAL ASSOCIATION.—President, E. T. Nesbitt; Secretary-Treasurer, J. H. Lauer, Montreal, Que.

CANADIAN ASSOCIATION OF STATIONARY ENGINEERS.—President, Wm. Norris, Chatham, Ont.; Secretary, W. A. Crockett, Mount Hamilton, Ont.

CANADIAN CEMENT AND CONCRETE ASSOCIATION.—President, Peter Gillespie, Toronto, Ont.; Secretary-Treasurer, Wm. Snaith, 57 Adelaide Street, Toronto, Ont.

CANADIAN CLAY PRODUCTS' MANUFACTURERS' ASSOCIATION.—President, W. McCredie; Secretary-Treasurer, D. O. McKinnon, Toronto

CANADIAN ELECTRICAL ASSOCIATION.—President, A. A. Dion, Ottawa; Secretary, T. S. Young, 220 King Street W., Toronto.

CANADIAN FORESTRY ASSOCIATION—President, John Hendry, Vancouver. Secretary, James Lawler Canadian Building, Ottawa.

CANADIAN GAS ASSOCIATION.—President, Arthur Hewitt, General Manager Consumers' Gas Company, Toronto; John Kelilor, Secretary-Treasurer, Hamilton, Ont.

CANADIAN INDEPENDENT TELEPHONE ASSOCIATION.—President, W. Doan, M.D., Harrietsville, Ont.; Secretary-Treasurer, Francis Dagger, 21 Richmond Street West, Toronto.

THE CANADIAN INSTITUTE.—198 College Street, Toronto. President J. B. Tyrrell; Secretary, Mr. J. Patterson.

CANADIAN MINING INSTITUTE.—Windsor Hotel, Montreal. President, Dr. A. E. Barlow, Montreal; Secretary, H. Mortimer Lamb, Windsor Hotel, Montreal.

CANADIAN PEAT SOCIETY.—President, J. McWilliam, M.D., London, Ont.; Secretary-Treasurer, Arthur J. Forward, B.A., 22 Castle Building, Ottawa, Ont.

THE CANADIAN PUBLIC HEALTH ASSOCIATION.—President, Dr. Charles A. Hodgetts, Ottawa; General Secretary, Major Lorne Drum, Ottawa.

CANADIAN RAILWAY CLUB.—President, A. A. Goodchild; Secretary, James Powell, P.O. Box 7, St. Lambert, near Montreal, P.Q.

CANADIAN STREET RAILWAY ASSOCIATION.—President, Patrick Dubee, Montreal; Secretary, Acton Burrows, 70 Bond Street, Toronto.

CANADIAN SOCIETY OF FOREST ENGINEERS.—President, Dr. Fernow, Toronto.; Secretary, F. W. H. Jacombe, Department of the Interior, Ottawa.

CENTRAL RAILWAY AND ENGINEERING CLUB.—Toronto, President, G. Baldwin; Secretary, C. L. Worth, 409 Union Station. Meets third Tuesday each month except June, July and August.

DOMINION LAND SURVEYORS.—President, Mr. R. A. Belanger, Ottawa; Secretary-Treasurer, E. M. Dennis, Dept. of the Interior, Ottawa.

EDMONTON ENGINEERING SOCIETY.—President, J. Chalmers; Secretary, B. F. Mitchell, City Engineer's Office, Edmonton, Alberta.

ENGINEERING SOCIETY, TORONTO UNIVERSITY.—President, J. B. Ritchie; Corresponding Secretary, C. C. Rous.

ENGINEERS' CLUB OF MONTREAL.—Secretary, C. M. Strange, 9 Beaver Hall Square, Montreal.

ENGINEERS' CLUB OF TORONTO.—96 King Street West. President, Willis Chipman; Secretary, R. B. Wolsey. Meeting every Thursday evening during the fall and winter months.

INSTITUTION OF ELECTRICAL ENGINEERS.—President, Dr. G. Kapp; Secretary, P. F. Rowell, Victoria Embankment, London, W.C.; Hon. Secretary-Treasurer for Canada, Lawford Grant, Power Building, Montreal, Que.

INSTITUTION OF MINING AND METALLURGY.—President, Edgar Taylor; Secretary, C. McDermid, London, England. Canadian members of Council:—Prof. F. D. Adams, J. B. Porter, H. E. T. Haultain and W. H. Miller and Messrs W. H. Trewartha-James and J. B. Tyrrell.

INTERNATIONAL ASSOCIATION FOR THE PREVENTION OF SMOKE.—Secretary R. C. Harris, City Hall, Toronto.

MANITOBA ASSOCIATION OF ARCHITECTS.—P ident, W. Fingland, Winnipeg; Secretary, R. G. Hanford.

MANITOBA LAND SURVEYORS.—President, George McPhillips; Secretary-Treasurer, C. G. Chataway, Winnipeg, Man.

NOVA SCOTIA MINING SOCIETY.—President, T. J. Brown, Sydney Mines, C. B.; Secretary, A. A. Hayward.

NOVA SCOTIA SOCIETY OF ENGINEERS, HALIFAX.—President, J. N. MacKenzie; Secretary, A. R. McCleave, Assistant Road Commissioner's Office, Halifax, N.S

ONTARIO ASSOCIATION OF ARCHITECTS.—President, C. P. Meredith, Ottawa; Secretary, H. E. Moore, 195 Bioor St. E., Toronto.

ONTARIO PROVINCIAL GOOD ROADS ASSOCIATION.—President, Major. T. L. Kennedy; Hon. Secretary-Treasurer, J. E. Farewell, Whitby Secretary-Treasurer, G. S. Henry, Oriole.

ONTARIO LAND SURVEYORS' ASSOCIATION.—President, T. B. Speight, Toronto; Secretary, L. V. Rorke, Toronto.

TECHNICAL SOCIETY OF PETERBORO.—Bank of Commerce Building, Peterboro. General Secretary, N. C. Mills, P.O. Box 995, Peterboro, Ont.

THE PEAT ASSOCIATION OF CANADA.—Secretary, Wm. J. W. Booth, New Drawer, 2263, Main P.O., Montreal.

PROVINCE OF QUEBEC ASSOCIATION OF ARCHITECTS.—Secretary J. E. Ganier, No. 5, Beaver Hall Square, Montreal.

REGINA ENGINEERING SOCIETY.—President, A. J. McPherson, Regina; Secretary, J. A. Gibson, 2429 Victoria Avenue, Regina.

ROYAL ARCHITECTURAL INSTITUTE OF CANADA.—President, H. C. Russell, Winnipeg, Man.; Hon. Secretary, Alcide Chausse, No. 5, Beaver Hall Square, Montreal, Que.

ROYAL ASTRONOMICAL SOCIETY.—President, Prof. Louis B. Stewart, Toronto; Secretary, J. R. Collins, Toronto.

SOCIETY OF CHEMICAL INDUSTRY.—Wallace P. Cohoe, Chairman, Alfred Burton, Toronto, Secretary.

UNDERGRADUATE SOCIETY OF APPLIED SCIENCE, McGILL UNIVERSITY.—President, W. G. Mitchell; Secretary, H. F. Cole.

WESTERN CANADA IRRIGATION ASSOCIATION.—President, Duncan Marshall, Edmonton, Alta. Permanent Secretary, Norman S. Rankin, P.O. Box 1317, Calgary, Alta.

WESTERN CANADA RAILWAY CLUB.—President, R. R. Nield; Secretary, W. H. Rosevear, P.O. Box 1707, Winnipeg, Man. Second Monday, except June, July and August at Winnipeg.