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THE CONSTANT ANGLE ARCH DAM

A NEW TYPE OF DAM IN WHICH THE ARCH TAKES THE GREATEST PORTION OF THE LOAD, EVEN CLOSE TO THE FOUNDATION.

THERE is a new type of arch dam to which more than ordinary interest is attached, on account of several new features introduced for the first time in the design. These features accomplish a double purpose. They introduce great economy and they also make it possible for the arch to take the greatest portion of the load acting as an arch even close to the foundation.

So far, the greatest objection of engineers to the use of a pure arch dam has been that this kind of dam as ordinarily built can not deflect enough at and near the bottom to take the load on the arch. Most of the load here has to be taken up by shear and cantilever action, and therefore material sufficient for this purpose must be provided. In the new type of arch the length of the upstream radius decreases at a more or less uniform rate from the crest towards the foundation. In the ordinary type of arch dam this length is kept constant, or in case the upstream face is provided with a batter, this length increases from the crest towards the foundation. That this difference in the length of the upstream radii of the two types has an important bearing upon the economies of the design should be easily realized, when it is considered that the thickness of the arch dam section is proportional to the length of the upstream radius at any elevation and that the crown deflection is practically proportional to the square of the length of the upstream radius. Therefore, the smaller the length of the upstream radius, the smaller the required thickness and the arch deflection. This is of especial importance towards the bottom of an arch dam.

Leaving for a later article the description of two particular dams of this type already in service, the general calculation of arch dams will be given here with especial reference to the type referred to.

In order to obtain a preliminary dam section for any given dam site the simple formula

$$t = \frac{P \times R_u}{q} \qquad - \quad - \quad - \qquad (1)$$

can be used for finding the thickness of a sufficient number of arch slices at different elevations; and by superimposing these slices upon each other the dam section can be formed. In this formula t equals the thickness of the dam in feet at any given elevation; P equals the water pressure in pounds per square foot; R_u equals the length of the upstream radius in feet and q equals the average stress in pounds per square foot of the area of the dam section (Fig. 1).

From (1) it is seen that the thickness, and therefore the area of the dam section varies in direct proportion with the radius. The volume of concrete in any arch dam is equal to the area of the section times the length of the mean arc. The length of the mean arc can be expressed as the length of the mean radius times the subtended angle in terms of

$$\pi \text{ or } V = \text{area} \times R_m \times 2\theta \qquad - \qquad (2)$$

where 2θ is the subtended angle.

The mean radius R_m equals half the width W of the span divided by the sine of half the subtended angle (Fig. 1). Thus

$$R_m = \frac{\frac{1}{2} W}{\sin \theta} \qquad - \quad - \quad - \qquad (3)$$

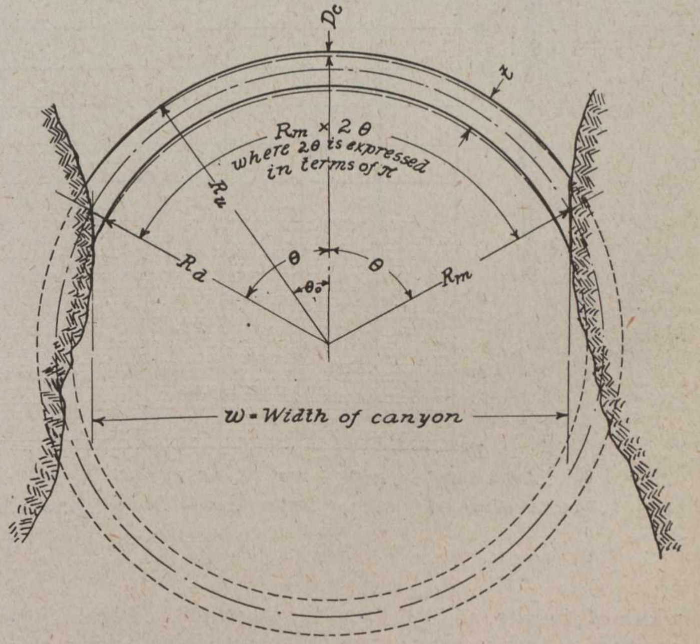

Fig. 1.

As the area of the section is proportional to the radius (both to R_u and R_m), (2), for the volume of masonry can be expressed thus:

$$V = C \times \frac{(\frac{1}{2} W)^2 \times 2\theta}{\sin^2 \theta} = \frac{K \times \theta}{\sin^2 \theta} \qquad - \qquad (4)$$

where C and K are constants, the latter depending upon the width of the canyon.

According to (4) the volume varies with the term $\frac{\theta}{\sin^2 \theta}$. The differential coefficient of this term equated to zero gives the minimum for a central angle of 133°, which means that any horizontal slice of the dam has the least volume when $2\theta \doteq 133°$. In other words, the dam contains a minimum amount of material when the central angle is kept 133° at all elevations.

The curve in Fig. 2 shows this graphically. The abcissas represent the central angle 2θ and the ordinates represent the term $\frac{\theta}{\sin^2\theta}$, the latter being proportional to the volume of masonry. In addition to showing the point of maximum economy, this curve also shows that as long as the subtended angle is kept above 110° the variation in the amount of masonry is very small, but below 110° the volume increases rapidly. Most all dam sites are narrower at the bottom than they are at the crest elevation; therefore, in order to place the material in the dam most economically, it is necessary to change the length of the mean radius of the dam continuously from the bottom to the top corresponding to the change in width of the site, so as to keep the subtended central angle constant. In practice it is seldom possible to keep this angle exactly constant. It is a mere ideal condition, but one should try to bring practice as close to theory as possible in designing the arch.

To prevent upper portions of the dam from overhanging lower portions, it will be necessary to have the thickness of the section increase from the crest towards the foundation. The proportional increase in water pressure must therefore be greater than the proportional decrease in length of the upstream radius towards the foundation. The ratio of increase in water pressure is always fixed, and the ratio of decrease in the length of the upstream radius depends upon the slope of the canyon sides. If these slopes are such, that at any intermediate elevation the ratio of decrease in length of the upstream radius has been greater than the ratio of increase in water pressure, a decrease in thickness of the dam at this elevation would result, and the structure would be overhanging, which is impractical.

Whenever a certain thickness must be provided to prevent overhanging, it is most economical to increase the length of the mean radius above that corresponding to a central angle of 133° for the reason that a flat arch requires less material than a more curved one of the same thickness.

In the foregoing the thickness of different arch slices at different elevations have been determined, from (1), as if all the load were taken by the arch and the dam had no gravity action at all. How safe a dam would result depends primarily upon the unit compression allowed when using (1) for finding the thickness at different elevations. This design, however, would in most cases prove to be weakest in the middle, for the same reason that a long column held at both ends is weakest in the middle and on account of having highest cantilever stresses here. Whenever t is small compared with R_u, the arch, when loaded, is practically a long column in compression, and the length of the arch should therefore not be over 25 times its thickness if the material is to be highly stressed. It is true that this circular column is supported to some extent along one side, but this added stiffness may be largely offset by the fact that the water may not soak through the upstream face uniformly, i.e., the effect of the water pressure would in all probability be unsymmetrical about the centre line of the dam. On a high, comparatively thin arch dam section, the resulting compression due to cantilever action and weight of material above may become excessive near the foundation requiring some additional material along the downstream face towards the foundation. The thickness of this added material should decrease vertically from a maximum at the foundation to zero at some higher ele-

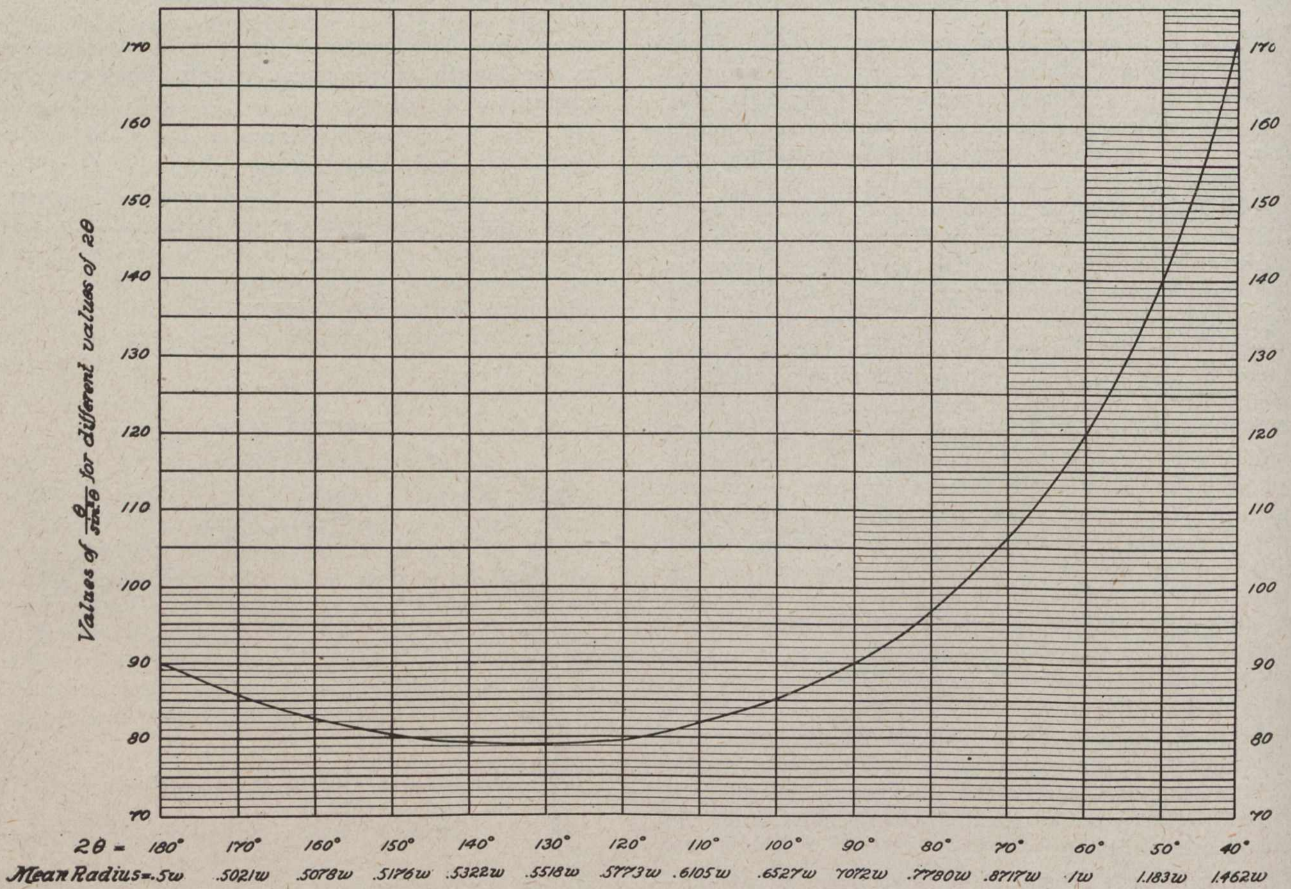

Fig. 2.

vation and horizontally from a maximum in the middle, or the point where the deflection is a maximum towards the abutments. The thickening of the dam in the middle to take care of cantilever stress also stiffens the arch materially, considering it as a curved beam. It acts as such to a large extent towards the foundation where t is large compared with R_u.

Before attempting to find what proportion of the load is carried by the arch and what proportion is carried by the cantilever it must be determined how much of the total load is carried by the initial stresses in the arch.

By initial stresses are meant stresses principally due to the weight of the structure and to the water pressure. Therefore, these stresses reach their maximum values at or near the foundation and are zero at the crest. They have not been much discussed so far, but are very important and should be taken into consideration when attempting to find the actual division of load between arch and cantilever. When a body is compressed the dimension in the direction of the compressive force becomes smaller, but in other directions the body swells if free to move (lateral strain). The ratio of lateral to longitudinal strain for concrete has been taken $\frac{1}{m} = \frac{1}{5}$ in the following calculations.*

Any horizontal layer of material will have to sustain compression corresponding to the height of masonry above it and will therefore actually become shorter in a vertical direction and have a tendency to expand horizontally. If the abutments are unyielding the arch may be prevented from actually becoming longer, in which case axial compression is introduced the same as if water pressure acted upon the structure.

If the specific gravity of the concrete for the dam is taken at 2·3 and the height of the dam H, then the average vertical pressure can be expressed as $\frac{2{\cdot}3\ H}{a}$; where a is the ratio of total height of dam to height of a rectangular wall having the same sectional area and the same base. The ratio a is known as soon as the section is known, and in dam design the section must be more or less determined before final calculation can be made.

The dam section in Fig. 3 has an area of 9,668 feet, a base width of 70 feet, and a height of 250 feet. The height of masonry column causing the mean vertical pressure is, therefore, $\frac{9668}{70} = 138$ feet and $a = \frac{250}{138} = 1{\cdot}81$; making the mean vertical compression upon the foundation in terms of head of water equal to $\frac{2{\cdot}3\ H}{1{\cdot}81} = 1{\cdot}27\ H$, with no water pressure upon the upstream side.

The condition of full reservoir introduces an additional force, viz., the radial water pressure, tending to compress the dam body in a direction perpendicular to the direction of the compressive force due to the weight of the body. At the bottom of the dam this force is equal to H in case the water is standing to the crest of the dam. In this case the radial water pressure tends to counteract the swelling of concrete in an up and downstream direction (due to the weight) thereby introducing additional initial axial compression. The total resulting initial axial compression at the foundation of section shown in Fig. 3 is therefore (in terms of head of water):

$$1/5\ (1{\cdot}27\ H + H) = 0{\cdot}454\ H \qquad - \qquad (5)$$

where Poisson's ratio has been taken equal to 1/5.*

The height of water, h, that this initial axial compression of 0·454 H will resist without causing any shortening of the length of the arch at the bottom can be found by using (1), thus:

$$h = 0{\cdot}454\ H \frac{t}{R_u} \qquad - \quad - \quad - \qquad (6)$$

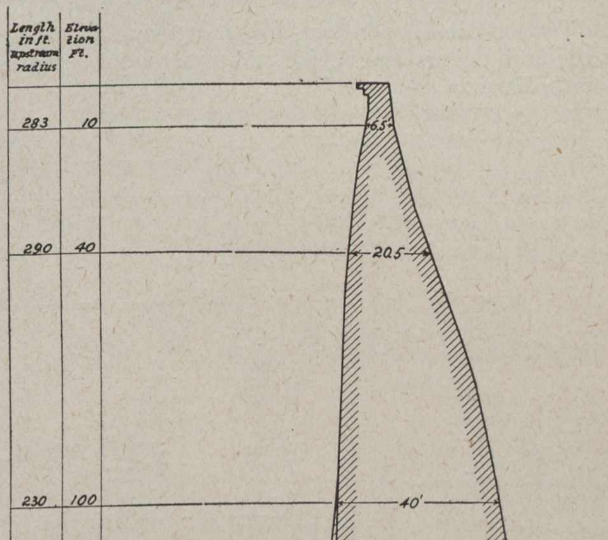

Fig. 3.

For the narrower section, shown in Fig. 3, $t = 70$ feet at the base and $R_u = 75$ feet. Substituting these values, it is seen that this section at the very bottom is able to carry $h = 0{\cdot}454 \times H \times \frac{70}{75} = 0{\cdot}425\ H$, or 42·5 per cent. of the total head of water as an arch before any shortening in the length of the arch occurs.

*Prof. C. von Bach has been kind enough to make some tests for the writer to determine m for concrete 1 : 2 : 3. He found for specimens 45 days old, using between 0.1 and 24 kg. per sq. cm. compression, m to be 5.3. See also Bach's Elastizitat und Festigkeit, 5 Auflage Seite 301. Considering that large stone will be embedded in the concrete in most dams the factor 5 has been used for m as probably representing most closely actual conditions.

*Mr. H. Ballet was probably the first to point out the necessity of taking Poisson's ratio into consideration when attemping to find the actual stresses in a dam body.—Proceedings of the Institute of Civil Engineers, 1909, Page 51.

The initial axial compression holds in equilibrium the stresses due to 42·5 per cent. of the total head at the bottom, the remaining 57·5 per cent. of the load will divide between cantilever arch and curved beam action in proportion to their relative carrying capacity.

By analyzing (6) it is seen that by simply varying t or R_u, or both, the designer can utilize more or less of the initial stress to carry the load. If the base thickness in Fig. 3 is increased from 70 feet to 110 feet and the thickness increased correspondingly at higher elevations, the initial stresses will be able to support at the foundation

$$0{\cdot}4 \times H \times \frac{110}{75} = 0{\cdot}585 \times H,$$ or 58·5 per cent. of the total water pressure before any shortening in the length of the arch occurs and before additional axial compression is introduced.

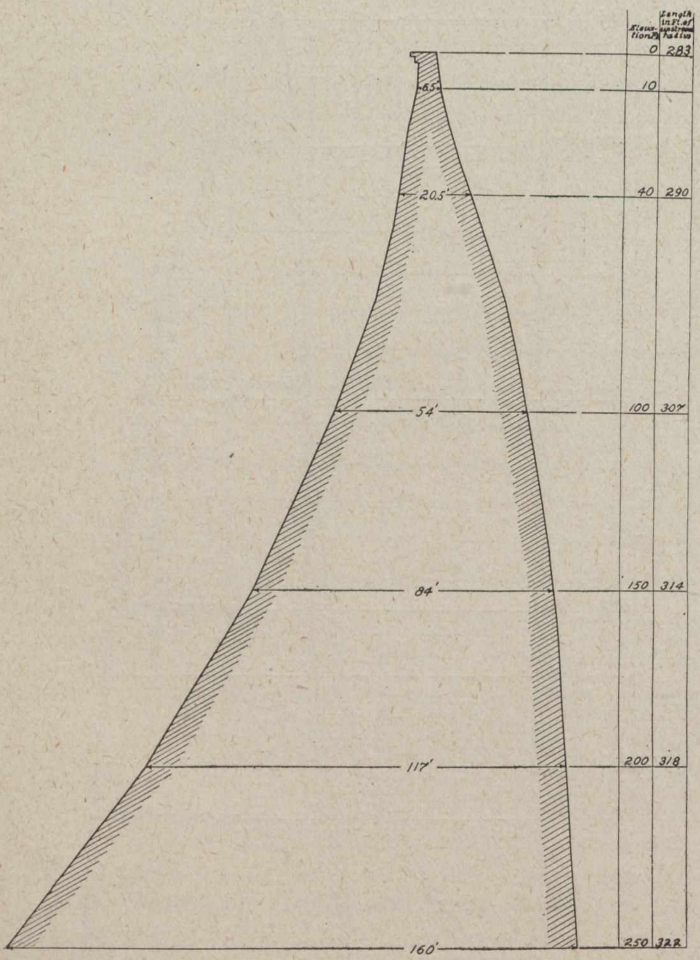

Fig. 4.

When the arch, however, becomes very thick in comparison with its length, the load is carried more by curved beam action than by ordinary arch action.

The dam shown in Fig. 3 was designed with varying radii to keep the central angle of the arch as nearly constant as possible at all elevations. For comparison, a section is shown in Fig. 4, using the same unit compression except where the section is wider than a gravity section near the foundation and the same upstream face batter, but a single common centre as ordinarily used for both upstream and downstream faces. For this section the length of the upstream radius is also variable, but it increases towards the bottom and reaches here a value of 322 feet. (See tables of lengths, Figs. 3 and 4.) The initial stresses in this dam will resist 20% of the head of the water at the bottom. It is, therefore, easily seen that the constant angle arch is much more effective in utilizing the initial stresses to carry the load than is the ordinary arch dam struck from a single centre. If a gravity section is insisted upon for the arch, but the central angle kept as near constant as practicable it will be possible for the gravity section to take up the greater part of the load acting as an arch and curved beam. The factor of safety has thereby been largely increased.

For finding the arch deflection the following formula has been used:

$$D_o = \frac{C\,C_o \times P_1\,(\text{upstream radius})^2}{E\,t} \qquad (7)$$

where $P_1 = p\,\dfrac{R_u}{R_m}$, and $C\,C_o$ is a factor which takes the curved beam action into consideration and can be directly found from Fig. 5.*

Formula (7) has been used for finding the deflection curves A and B (Fig. 6) of the two sections, Fig. 3, (base 110 feet), and Fig. 4. Formula (6) has been used for correcting these curves A and B to take the effect of lateral strain into consideration. These curves represent the deflection of the two arches assuming they are free to move at the foundation. They are plotted to show how evenly the deflection, curve A, slants from a maximum near the top to nearly nothing at the bottom in the constant angle arch type, Fig. 3, and how little the slant, curve B, amounts to in the ordinary type of arch dam, Fig. 4. These curves also show very plainly that from the common arch type much arch action towards the bottom cannot be expected; cantilever and beam action must take the load since no such deflection as 0·2624 in. could be possible at the point where the arch is fastened to the rock foundation. The constant angle arch type for this particular site requiring only 0·0083 in. deflection, 31·5 times less to support the same load will take most of the load upon itself acting as an arch.

For dam sites where the abutments are close together towards the foundation and where t is large compared with R_u, (7), gives the values for the crown deflection which are too large, even assuming that the dam is entirely free to move at the bottom. While this formula considers the curved beam action, it is at the same time understood that arch action is complete. However, where the arch is thick and the distance between the abutments short, the arch becomes a wedge and the horizontal curved beam takes the greater proportion of the load, as acting in this manner the support of the same load will require a smaller deflection. The deflection in the middle of a beam 1 foot wide held at both ends and uniformly loaded is:

$$D_b = \frac{P l^4}{384\,E\,J} \qquad (7A)$$

The notations are the same as before, P being the unit water pressure, l the length of the beam, E the modulus of elasticity of concrete and J the moment of inertia using like units.

Whenever (7A) gives smaller values than (7) it is indicated that arch action is incomplete. The curved beam action tends to introduce axial tension along the downstream face in the middle and along the upstream face near the abutments, but the axial compression due to the partial arch action and lateral expansion (Poisson's ratio) will or should much more than compensate for this tendency. If it does not, the design should be changed.

*This Fig. and Formula (7) are reproduced from a discussion by Mr. Shirreff of a paper entitled, "Lake Cheesman Dam and Reservoir," Transaction American Society Civil Engineers, 1904, page 89. E is the modulus of elasticity and t is the thickness.

From curve A the deflection of the arch (Fig. 3) at the ⅓ point can be directly ascertained. It is found to be 0·132 in. If the cantilever 250 feet high and 1 foot wide were actually forced to deflect 0·132 in. at this point (Elev. 166·67 feet) a force, F, would be required which can be found as follows (F is concentrated at the ⅓ point):

$$D_o = \frac{n\,F\,l^3}{E\,h^3} \quad - \quad - \quad - \quad - \quad (8)$$

In (8) (taken from standard handbooks) the value of n depends upon the rate of variation of the face slopes. If both faces were vertical n would equal 4. If the faces (or at least the downstream face) were shaped as flat parabolas, or if the thickness of the section in an upstream and downstream direction at the one-third point was approximately half the thickness at the foundation, n would equal 8.

This last condition is the one that theoretically best fits cases in dam construction. Considerable modifications are mostly necessary, however, due to the fact that the rock foundation itself, to some extent, takes part in the movements of the dam body. With a full water load the rock foundation under the middle portion of an arch dam moves more in a downstream direction than does the ends, as the push in a downstream direction is the greatest in the middle and as at the ends, only a component of the axial compression acts in a downstream direction. Therefore, the cantilever can not take up as great a proportion of the water load as it would if fastened to an immovable foundation and more load is therefore thrown on the arch. The writer has for some time been trying to find a practical value for n by analyzing deflection data obtained from actual dams. He thinks he is justified in using $n = 12$ for solid rock foundation, and 16 for seamy rock foundation. This makes (8) empirical, but the results from it are believed to be closer to actual facts than any results arrived from mere theoretical conditions on account of the number of assumptions it is necessary to make.

Inserting the value of 12 for n in (8):

$$D_o = \frac{0{\cdot}132}{12} = \frac{12 \times F \times 83{\cdot}33^3}{432{,}000{,}000 \times 110^3}$$

$$F = 911{,}000 \text{ lbs.}$$

The cantilever will deflect the same as the arch when thus loaded.

The total water load on a vertical slice of the dam, 1 foot wide and 250 feet high, is $250\,\dfrac{0{\cdot}15{,}625}{2} = 1{,}953{,}125$ lb. The initial stress supports $1{,}953{,}125 \times \dfrac{16{\cdot}4}{100} =$ 320,312 lb. before any deflection takes place. Therefore the load causing a deflection of 0·132 in. of the combined arch and cantilever must be equal to 911,000 + 1,953,125 — 320,312 = 2,543,813 lb. The proportion of this amount taken by the cantilever will be $\dfrac{911.000}{2{,}543{,}813} = 35{\cdot}8$ per cent.

Now, the actual load to be divided between cantilever and arch is not 2,543,813 lb. per running foot, but only 1,953,123 — 320,312 = 1,632,813 lb. Of this amount the cantilever carries 35·8 per cent., or $1{,}632{,}813 \times \dfrac{35{\cdot}8}{100} =$ 584,547 lb., concentrated at the one-third point, making the actual deflection at this point $0{\cdot}132 \times \dfrac{64{\cdot}2}{100} = 0{\cdot}0847$ in.

The bending moment due to this force is equal to 584,547 × 83·33 = 48,710,301 ft.-lb.

The section modulus of the base $= \dfrac{110^2}{6} = 2{,}011$, and therefore the compressive stress on the foundation at the toe, due to the bending action of the water load on the cantilever, is equal to

$$\frac{\text{Bending moment}}{\text{Section modulus}} = \frac{48{,}710{,}301}{2{,}011} = 24{,}222 \text{ lb. per sq. ft.} \quad (9)$$

The total compression on the foundation at the toe will be this compression added to that due to the weight of the structure, which amount to approximately 16,200 lb. per sq. ft. at the toe, making the total compression approximately 40,400 lb. per sq. ft.

If a base length of 70 ft. is chosen, the arch would take a greater percentage of the load and the curved beam a smaller, leaving the same or less for the cantilever, but, owing to the smaller section modulus of the 70-ft. base, the compression at the toe would be somewhat higher than 24,222 lb. per sq. ft., and the compression due to the weight of the structure would be much higher than

$$CC_c = \frac{\frac{2\sin\theta}{\theta}(1-\cos\theta)+\frac{1}{2}(\cos 2\theta-1)}{\frac{3\theta}{\sin\theta}+\cos\theta-4}+(1-\cos\theta)$$

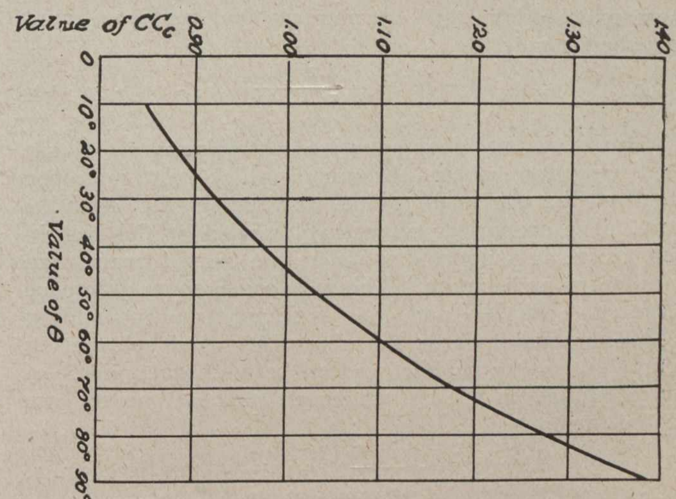

Fig. 5.

16,200 lb. per sq. ft., so that the sum of the two would be considerably more than 40,400 lb. per sq. ft. Although within the safe limit, the resulting vertical compression would be somewhat out of proportion to the 36,000 lb. per sq. ft. (and less) axial compression used when calculating t from (1).

The dam section with the 110-ft. base contains only 4% more material than the dam with the 70-ft. base (Fig. 3), as the addition is not made as a portion of a circular ring, but in the shape of a spherical triangle. Any intermediate base length between the two limits given in Fig. 3 could be accepted for a dam built on this particular site. The two stresses (the 36,000 lb. per sq. ft. average axial compression, and the maximum 40,400 lb. per sq. ft. vertical compression) are acting in planes perpendicular to each other, and therefore tend to support each other. Although they are low, the resulting section (Fig. 3) appears slender on account of the economical distribution of the material.

This method of calculating the vertical stress upon the foundation is correct only so long as no tension exists at the heel, or if tension exists, as long as this tension is properly taken care of. For the constant angle arch, where the cantilever takes the smaller proportion of the load, there will seldom be occasion for tension along the upstream face, and there will perhaps never be enough tension to demand consideration. The accuracy of the result obtained from 8 depends to some extent upon the face slopes, especially the downstream face slope. The error, however, is generally such as to compensate for the error made in not considering that the width of the vertical cantilever, which is 1 foot at the upstream face, is less at the downstream face. The short-cut method explained above for finding the division of the water load between cantilever and arch action, and from that for finding the total maximum foundation pressure, cannot be used for dams having a crown deflection curve similar to line B (Fig. 6), as this line does not show a maximum deflection near the crest, and a zero deflection at the foundation. Deflection curve A answers these conditions closely enough for this purpose.

Only the middle or highest dam section has been considered, as we are mostly interested in knowing the most dangerous stresses in the structure, which stresses generally occur, in high dams at least, at the toe with reservoir full.

In (1) only average stresses have been considered in determining the thickness of each individual arch slice. The maximum axial stresses should also be investigated. These exist along the downstream face and are found from the formula,

$$Q \max = q \frac{2 R_u}{R_u + R_d} \qquad (10)$$

However, (10) does not give correct results towards the foundation where the arch is thick relative to the length of the upstream radius, and where the span is short. The proportion of the load carried by the arch in such a case is supported more by the curved beam than by ordinary arch action. This will cause some difference in the value of Q_{max} and q_{min}, as found from (1), adding to Q_{max} at and towards the abutments, and subtracting from Q_{max} in the middle portion between the points of contra flexure on the curved beam. These points are located thus:

$$\operatorname{Cos} \theta_o = \frac{\sin \theta}{\theta} \qquad (11) \text{ See Fig. 1.}$$

In high dams Q_{max} will ordinarily be lower than the vertical compression at the toe; therefore, this vertical pressure is still the most important to investigate. The influence of initial stress (Poisson's ratio) tends to equalize Q_{max} and q_{min} in dam sections having upstream faces of steeper slope than their downstream faces. In such sections the vertical pressure due to the weight of material above is greatest along the upstream face, and therefore the initial axial compression is also greatest. It is fair to assume that this condition of relieving Q_{max} and adding to q_{min} also tends to improve the watertightness of the dam.

In all straight gravity dams built across narrow canyons, horizontal tension exists along the downstream face in the middle, and along the upstream face near the abutments, at least toward the foundation. This should be very plain when it is considered that any beam fixed at both ends and uniformly loaded will support four times as much load as a cantilever of the same length sustaining the same water load (nothing at the top and a maximum at the bottom). In other words, whenever the beam is four times longer than the cantilever, it will support one-half of the total load, and whenever this ratio is less than four, the horizontal beam will support most of the load. The ordinary gravity design does not consider this beam action, although when the dam is built in a fairly narrow canyon the greatest portion of the load towards the foundation is actually carried on the horizontal beam and not on the cantilever. While adding materially to the stability of the dam (as long as the horizontal tension introduced by this beam action is not above the breaking

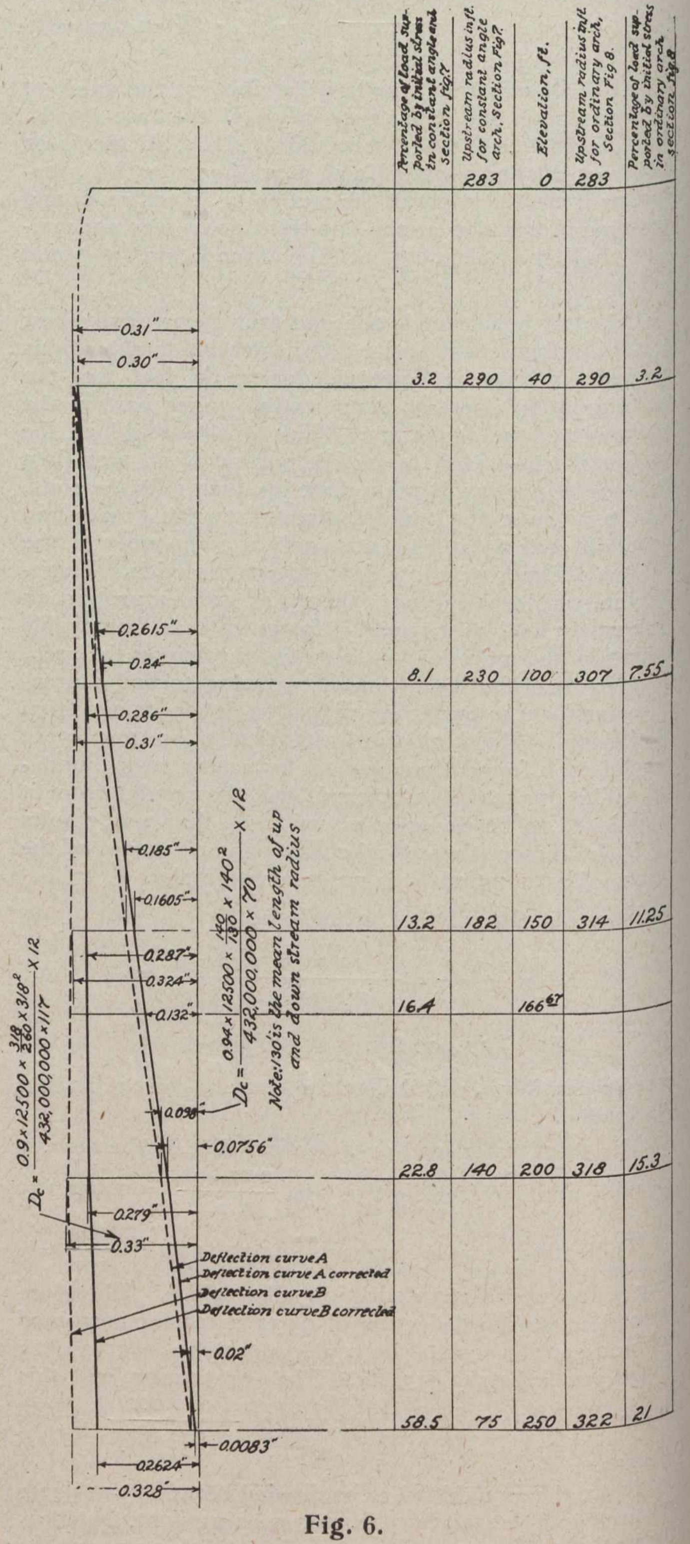

Fig. 6.

point, and as long as the expansion joints, if any, are placed at or near the points of contra-flexure only) the foundation pressure at the toe is at the same time much relieved, a very welcome feature, especially in connection with high dams.

Now, if the horizontal beam be curved, axial compression takes place over the entire section, and the greater the curvature, that is, the smaller the length of the up-stream radius, the more load will be taken by the arch, and the less remains to cause horizontal axial tension at any point of the dam faces, due to beam action. The resultant axial compression from arch action and lateral expansion will, in general, more than compensate for this tension. Lateral expansion due to the weight of the structure exists, of course, whether the dam is straight in plan or curved, but this alone will seldom be sufficient to compensate for the horizontal tension due to beam action in a straight gravity dam across a narrow canyon. The curvature must be introduced to be sure of no tension existing in this horizontal beam. For a dam 250 feet high the bottom width of the canyon would have to be well towards half a mile before a gravity dam would act simply as a gravity section towards the bottom, and before the influence of the horizontal beam action would be negligible unless it should have failed in tension first.* It would, therefore, seem logical to provide even quite long dams with a slight curvature sufficient to take care of the horizontal tension.

Whenever load is supported on a beam or on a cantilever, shearing stresses are introduced. These shearing stresses reach their maximum values at the foundation and at the abutments, and should be investigated in order to be sure that they are within safe limits. In the case of dam section shown in Fig. 3, base 70 feet, it can easily be shown that even should the shear on the lower 50 feet of the dam correspond to the full water pressure, this stress would be entirely within the safe limits, amounting to approximately 4,000 pounds per square foot. Along this joint the maximum unit compression generally exists. This compression is so much larger than the shear that actual shear cannot take place, as friction alone will prevent any tendency of sliding at the abutments. Crushing would have to take place before actual sliding of any element could occur.

Before concluding the general discussion the action of changes in temperature should be considered. As the ends of the arch are fixed to the abutments, the shortening or lengthening of the arch due to temperature changes either causes cracks to develop or forces the crown back and forth. In the constant angle arch type the average crown deflection for the same amount of decrease or increase in length or arch is only about one-half of that required by the common arch type, (See curve A and B, Fig. 6) and the tension or compression necessary to cause this deflection or pulling back of cantilever may therefore not exceed the ultimate strength of the concrete, in which case no cracks would develop. In any event cracks are not so liable to occur as in the common arch type of dam.

*Near the top the horizontal beam would have no practical influence.

The Quebec Government is determined to reduce the number of forest fires in the Province, and legislation with this end in view has been introduced by the Honourable Jules Allard, Minister of Lands and Forests. The changes about to be made, briefly summarized, follow: First, that all persons between the ages of 19 and 50 may be called on to aid Government officials in fighting fires, and must respond to the call of the Government in such cases, unless they can submit valid reasons for refusing to serve as fire fighters. For each day they work for the Government in fire fighting they will be paid $1.50 to $2. Another amendment aims to reduce the danger of fires from railway trains. It will be required that timber limit holders shall clear the trees from each side of railway tracks for at least a distance of 100 feet. The third change provides that settlers will be prohibited from clearing timber in summer months without a permit from the Government forest ranger.

MAPS AND PLANS FOR HIGHWAY ENGINEERS AND SUPERINTENDENTS.*

By T. M. DeBlois, S.B., Assoc.Mem.Can.Soc.C.E.

ONE of the important divisions of highway engineering, and in fact of any branch of engineering, is the accumulation, the proper classification and the convenient recording of data of interest or importance to the engineer.

Maps and plans, in connection with a highway organization, might be classed in the service department. No matter how thoroughly the operations of a highway department may be systematized, where those in charge of the different units of the organization are dependent upon daily consultations and studies of the records on file to enable them to picture in their minds the progress of the work under their jurisdiction, there is always a certain amount of lost motion which would be eliminated if all available information and a continuous record of operations were indicated on properly prepared maps and plans.

As road maps, the original township survey maps, at a scale of one-half mile to the inch, filed with the Provincial Survey Branch, are of little use, as the great majority were prepared long before many of the roads were in existence. These maps, however, do give the frontage and area of the lots, and naturally form the background of other maps which have later been prepared, showing the divisions of the land into concessions and lots.

In connection with the township maps of Ontario, we notice that many have more or less regular systems of jogs along the side roads, which follow similar systems of jogs in the lot lines midway between the concessions. This irregularity is due to the inaccuracy of the compass survey. For example, the surveyor runs his line and places his posts at regular intervals along the concession road, going from one concession road to the next. If his chainage were accurate and his lines run truly parallel, a lot line run at right angles to the concession road would obviously hit the corresponding post on the adjacent concession road. However, when a later surveyor stakes out the lots on the ground he finds that lines run from each concession line at right angles do not meet at the centre of the concession, and to rectify the error the lot line jogs to make a closure. Still later, when the side road is laid out, it must either make the corresponding jog or run through the property of the lot owner.

However, the most complete road maps of the whole province are those issued by the Department of the Interior at Ottawa, known as the Standard Topographical Maps. The maps are at a scale of 3.95 miles to the inch, and show the plan of the province from Windsor to Montreal in sheets about 28 inches wide. These sheets show the roads in double line as accurately as can be shown at such a small scale.

While the standard topographical maps of the Department of the Interior have been referred to as the most complete for the whole province, the one-mile-to-the-inch maps of the Department of Militia and Defence covering that portion of Ontario from Kingston and Ottawa to the eastern boundary of the province, and from Toronto and Buffalo to the western boundary, supply us the most accurate information regarding the limited portion they cover, that are at our disposal. These maps locate all roads and streets, lakes, rivers and creeks, bridges and

*Read before the Conference on Road Construction, Ontario Department of Highways.

culverts, buildings, wooded areas and swamps, etc., etc., and are traversed by contour lines at 25-foot intervals.

The military maps are dependent upon surveys conducted by that department, to as fine a degree of accuracy as can be reproduced on a map at the scale of one mile to the inch. The control, which goes to make up the framework of the maps, is procured from triangulation, chain, transit traverses, transit traverses with bicycle along railways, and stadia transit traverses, and is based upon points established in Canada along the shores of the Great Lakes and River St. Lawrence by the United States Coast and Geodetic Survey; also geodetic surveys conducted by the Department of the Interior at Ottawa.

Our Department of Highways has prepared a complete set of county maps of Ontario at the scale of one mile to the inch, showing roads in double line, lots, rivers, railways, etc., and also practically a complete set of township maps at double the size of the county maps, namely, one-half mile to the inch. These maps have been compiled from the best available sources and are being corrected as later information is obtained.

As a suggestion of a few of the uses to which these maps can be put, the following is submitted: First of all it would be advisable to adopt a standard legend to show the exact character of every piece of work locating the county road system, whether concrete, macadam, gravel or earth which has been only shaped and crowned or which has not been shaped or crowned or otherwise improved. A standard legend should also be prepared which will show the character of construction of every sluice, culvert and bridge, which should be plainly numbered, and these numbers used in all reference, by correspondence or otherwise, to these structures.

The matter of standardization of this legend is important. The maps in possession of the various branches should be so characterized that the information contained on them can be immediately read by any engineer familiar with the legend.

A further use to which the maps can be put is to mark on them the history of the roads in the various townships. For instance, the year in which the road was opened, the nature of the material, when the improvements were made, and the nature of the improvement. The history also could be given of the various important culverts and bridge structures. Those structures which had depreciated to the extent where it would be advisable to have them replaced could also be shown.

All these uses that can be made of county and township maps, some of which have been suggested, others which may have special application to particular localities, will perform a distinct service to the road engineer or superintendent. They will serve to keep before him in a manner that should be comprehensive and clear, a complete picture of the history and actual conditions of the highways in his particular locality. By so being in touch with the men who are familiar with the different localities, the department will thus be enabled to improve and bring to a higher state of accuracy our township and county maps. This point is important, for, while many of our maps are obtained from accurate sources, yet with many of them there are necessarily errors both of "omission and commission" due to a scarcity of information at our disposal, and which, by the means suggested, would thus steadily be improved.

For the opening or extension, or indeed the improvement of roads, the importance of preparing a plan and profile of the right-of-way cannot be over-emphasized.

If a man proposed to build himself a house, he first of all considers as a self-evident necessity, the preparation of complete plans and details of his proposed dwelling. This in order, first, that he may estimate his costs, and secondly, that he may have the processes before him of the construction from the cellar to the roof of his finished building. For similar and very obvious reasons, the building of a highway should be accompanied by a plan and profile of the proposed right-of-way. It further would be a valuable record to the county engineer to keep a profile of the more important roads in his county. With this record he can see at a glance the various hills and grades, and in the event of improving that road and incurring the reduction of some of the grades, proper estimates can be prepared and the problem can be approached in a systematic manner.

SOME HINTS ON THE APPLICATION OF SEWAGE TO CONTACT BEDS.

In the annual report of the New Jersey State Board of Health, Francis E. Daniels describes the methods of applying sewage to contact beds as practised at Essex Falls and Plainfield. At these plants the tank effluent is applied on the top and at one corner of the contact beds. From a small area at the point of application, from 6 in. to 1 ft. of the contact material is removed from the top of the beds and the excavation is filled with fine cinders. An embankment about a foot high is constructed of the same material around this area so that all the tank effluent that is applied to the contact beds strains through the cinders. A great deal of suspended matter is thus removed from the tank effluent, and the clogging of the body of the bed is materially reduced with a corresponding increase in the effective life of the contact beds.

With the growing practice of reducing the storage capacity of sedimentation tanks, the value of underfed contact beds is diminishing. With a long period of tank storage, the tank effluent is usually very septic and contains considerable quantities of offensive gases. With a tank effluent of this nature, odors are probably reduced by having contact beds of the underfed type. With the present practice of constructing sedimentation tanks so that the storage period is reduced, the effluent as a rule is not septic, and offensive gases are not present to any great extent. In such a case the overfed contact bed is preferable to the underfed type.

In the overfed type of contact bed it is maintained that the use of distribution troughs or pipes to distribute the sewage over the surface of the beds is inadvisable. These troughs distribute the clogging suspended material over the whole surface of the beds, and sometimes require considerable cleaning and attention.

The cinder straining areas not only prevent, to a large degree, the clogging of the whole surface and body of the bed, but they serve as an index of the operation of the sedimentation tanks. The scum accumulating on these areas can be removed from time to time between doses. The rapidity with which the scum forms will serve to indicate to the attendant whether or not the tanks should be cleaned.

An interesting point was brought up recently in the Iowa Supreme Court, when the right to use water in a reservoir or pond conveyed to a railroad company was reserved to the land owner. The latter's animals so polluted the water that it was unfit for use in locomotives, the purpose for which the railroad desired it; and the railroad fenced the pond. The landowner sued in equity to enjoin the fencing and for damages. The court holds that in such a case each party must exercise his rights with reference to the rights of the other, and the landowner could not allow his stock so to pollute the water. The railroad, however, could not exclude the landowner from all use, but should apply to the courts to restrain the pollution.

WOOD AS A PAVING MATERIAL.*

By W. Kynoch,
Chief Assistant, Wood Preservation Department, Forestry Branch, Department of Interior, Canada.

DURING the past decade there has been a steadily increasing interest among municipal highway engineers in the possibilities of creosoted wood block paving for city streets. The history of creosoted wood block paving has not been free from failures, but it may be noted with satisfaction that these failures have been made the subject of careful observation and experimental study, not only by those associated with commercial interests, but also by independent technical investigators. As a result, the causes of many of these early difficulties have been located, whether in methods of treating blocks or in design and construction of pavement, and improvements in practice have been made accordingly.

It may be of interest to quote from statistics compiled and published by the United States Forest Service. In 1909, the total amount of timber treated as paving blocks for use in cities of the United States was 2,994,290 cu. ft., equivalent approximately to 1,150,000 sq. yds. of pavement. In 1914, the timber treated for this purpose was 6,869,370 cu. ft., equal to about 2,617,000 sq. yds. In 1911 the reported area of creosoted wood block pavement in service in a number of the larger representative cities of that country was as follows: New York, 650,000 sq. yds.; Chicago, 700,000 sq. yds.; Minneapolis, 950,000 sq. yds.; Indianapolis, 500,000 sq. yds., and Cincinnati, 375,000 sq. yds.

Referring to the present use of wood block paving in England, and more particularly in London, as the world's greatest metropolitan district, it is interesting to note that creosoted wood block paving has there reached its most successful development. The significance of this statement can be appreciated fully when the traffic conditions of some of the principal thoroughfares of London are understood. Henry W. Durham, chief engineer of the Bureau of Highways, Borough of Manhattan, New York, commissioned in 1913 to make a personal investigation of paving materials and pavement construction and maintenance in European cities, in a report recently published, notes with reference to the wood block pavements of London, ". . . a large extent of soft-wood pavements on its principal thoroughfares. (Borough of Westminster). The last are probably the finest pavements in the world. Particularly good is that on Parliament Street and Whitehall from Parliament Square to Trafalgar Square. It carries a heavy traffic, principally motor omnibuses and other motor vehicles. Having less traffic but a very extensive one of pleasure vehicles of all classes is the Mall, extending from the Admiralty Arch at Charing Cross to Buckingham Palace. This is also wood block on concrete foundation and presented the nearest approach to a perfect street surface observed anywhere."

Untreated wood block pavements were in use in England at least eighty years ago. In the United States and Canada such pavements, generally built of round cedar blocks, were quite widely used as early as 1850, and some cases of such pavements were to be found as recently as fifteen or twenty years ago. These blocks were laid on plank foundation or in some cases on macadam foundation only. These pavements obviously could not be entirely satisfactory, but they served a useful purpose during a certain development period in the United States and Canada. Later, untreated rectangular wood blocks were adopted. This was an improvement in some respects, but early failure of such pavements from decay of blocks was the inevitable result. Later the development and more general use of preservative treatment of wood suggested the application of such treatment to wood paving blocks.

According to first methods of treatment adopted for this purpose, blocks were dipped in hot creosote oil. Such treatment resulted in the absorption of from two to four pounds of oil per cubic foot of timber. This marked a distinct improvement in practice, and such dipped wood pavements may be considered the immediate step toward the development of modern creosoted wood block paving. Considerable areas of dipped block pavement were laid in United States and Canada with satisfactory results. There are such pavements still in service in Canadian cities, many of which are now in good condition. It may be of passing interest to note that there are several dipped wood block bridge floors in the city of Ottawa, laid from six to eight years ago, which have given good service, and are at present in very satisfactory condition. However, the general adoption of pressure methods for impregnating timber with preservatives naturally led to the use of pressure treated paving blocks. Absorptions of creosote up to 20 pounds per cubic foot may be obtained by such methods, and with the heavier impregnation of preservative, the protective value of the treatment is obviously very much increased. The creosote oil injected within the block serves the double purpose of protecting the wood from decay and acting as a waterproof filling material. Pressure treated blocks are now used universally for wood paving, and it should be understood that claims made in this paper for creosoted wood block paving, refer to such methods of treatment.

A brief description of the present commercial methods of treating paving blocks may be of interest. Blocks are cut from 3-in. or 4-in. stock, varying in width from 5 ins. to 10 ins. The stock is planed on one side to insure a straight edge, and is cut on gangs of small circular saws on which provision is made for adjusting saw spacing, according to the depth of blocks required. In American practice, blocks are cut in 3-in., 3½-in. and 4-in. depths, according to the requirements of traffic conditions for which they are to be used. The sawn blocks from the gang pass to conveyer, where defective block—those under size or showing heavy checks, loose knots or decay —are removed. Inspected blocks are carried by conveyer to cylinder block cars. These block cars are from 7 ft. to 9 ft. long, of cylindrical shape, 5 ft. to 6 ft. in diameter, mounted on narrow-gauge trucks. Cars are made with perforated steel plate sides and closing cover to prevent blocks floating out in retort. Commercial pressure treating retorts or cylinders are usually 6½ ft. to 7 ft. in diameter and from 120 ft. to 130 ft. long. They are designed for working pressures up to 250 lbs. per square inch, although operating pressures during treatment of timber seldom exceed 160 lbs. to 175 lbs. per square inch. Retorts are charged with a train of loaded block cars. The capacity of a commercial retort of typical size is from 1,800 to 2,100 cu. ft. of paving blocks, equivalent approximately to 600 to 700 sq. yds. of 4-in. block pavement.

After blocks are charged into retort they may be subjected to a preliminary steaming at temperatures ranging from 225 to 240 degrees F., for a period of from 1 to 3

*Abstract of a paper read January 19th, 1916, at Ottawa, before the Canadian Lumbermen's Association.

hours, as may be required. If this feature of treatment is adopted, it must be followed by a vacuum drawn in the retort up to 20 ins. or 24 ins. mercury. Exhausting the retort after steaming is for the purpose of evaporating the water and withdrawing a part of the air contained within the wood. Preliminary steaming and vacuum before the admission of the preservative to the retort, is not always specified in the treatment of paving blocks. However, results of an experimental investigation by C. H. Teesdale at the United States Forest Products Laboratory, Madison, reported in a paper presented at the eleventh annual convention of the American Wood Preservers' Association, January, 1915, indicate that these features of treatment are desirable for blocks of some species at least. After steaming of blocks and exhaustion of the cylinder, or in some cases, as above noted, without any such preliminary treatment, creosote oil, previously heated to a temperature of from 170 to 200 degrees F., is admitted to the retort, the pressure applied gradually until the desired absorption of preservative has been attained, as indicated by gauges on oil stock tanks. The oil is then withdrawn from the retort and, if desired, a subsequent short steaming period may be included in the treatment, for the purpose of cleaning the surface of the blocks. A final vacuum, after the injection of the preservative, for the purpose of withdrawing excess of oil from the wood, is a feature of one proprietary method of treatment which has been extensively used in United States and Canada with good results.

The preservative used almost universally for paving blocks treatment is coal tar creosote oil—either a straight distillate product of coal gas or coke oven tar, or more generally, in present commercial practice, a mixture of such distilled oil and filtered tar. These mixed "paving oils" are now extensively used for paving block treatment in America, and it is probable, if they meet the requirements of approved detailed specifications as to methods of production, composition and physical properties, that they are as satisfactory for this purpose as the higher priced straight distilled oils. The absorption of oil in paving-block treatment, with the exception of Douglas fir blocks, is generally specified to be from 16 to 20 lbs. per cubic foot in correct American practice. While absorptions of 20 lbs. per cubic foot were formerly generally required, the recent general tendency is toward somewhat lighter impregnations—16 to 18 lbs. per cubic foot. The writer is inclined to believe that absorption of from 14 to 16 lbs. per cubic foot are sufficient for some Canadian woods for which heavier treatments have heretofore generally been specified. An absorption of 10 to 14 lbs. per cubic foot is the usual standard for the treatment of Douglas fir blocks.

The design and methods of construction of wood block pavements are in general similar to those of brick or other block pavements, although certain features have been developed to meet the special conditions. A concrete base is laid 5 ins. to 6 ins. in thickness, conforming in general to the contour of the finished pavement, usually with a crown allowance of 8 ins. on a 50-ft. street width. On this a ¾-in. cushion of a 1 : 3 cement-sand-mortar is spread dry and struck off with a template or spacing strips. This cushion is sprinkled in advance of the placing of the blocks. These are laid with the grain vertical, in straight paralleled courses, which may be at right angles or inclined to the street line. A ¾-in. or 1-in. expansion joint is allowed at each curb and a single or double course of header blocks placed parallel to the curb line. After laying, blocks are rolled to surface with a steam roller, and block joints filled with a suitable bituminous filler material. This is generally specified to be an asphalt cement or a mixture of coal tar pitch and asphalt. This material must be carefully squeegeed into joints to a depth of at least ⅔ the depth of blocks. The finished pavement is covered with a light layer of clean sand, which cleans the surface of the excess of filler and oil, and incidentally is partially worn into the blocks by traffic, forming a resistant wearing surface. A thin cement grout washed over pavement after pouring bituminous filler is frequently used as an additional means of securing a clean surface.

It is unnecessary within the limits of this paper, to offer any more detailed description of methods of treatment of wood paving blocks or design and construction of pavement, all of which are covered in carefully developed specifications adopted by various municipal and highway engineering associations. Such approved specifications may be safely adopted, either entirely or with modifications to meet local conditions.

The initial cost of wood block paving is fairly high as compared with some other widely used types of pavement. The cost of completed pavement will range from $2.50 to $3.80 per sq. yd., varying with the depth of block used, and with local conditions. Actual cost figures from construction of wood block pavements in Eastern Canada are: $2.75 to $3 per sq. yd. for 3-in. block pavement; $3 to $3.40 per yard for 3½-in. pavement, and $3.25 to $3.70 where 4-in. blocks are used. On the Pacific Coast the cost of wood block pavements is somewhat lower than that indicated by the above figures. Transportation charges on treated blocks and the cost of actual pavement construction are the more important variable factors, and these may be estimated closely for any particular local condition. However, admitting the relatively high first cost of wood block paving, it must be realized that ultimate cost, estimated on service performance, and not initial cost only, is the fair and logical basis of comparison of various paving materials.

With reference to the various wood species which have been successfully used for creosoted wood block paving, it may be noted that a considerable number of woods have been proved to be adapted to such service. In England, Baltic pine has been used most extensively and with excellent results. In the United States, the woods now in general use for paving are southern yellow pine, Norway pine, Douglas fir, hemlock, tamarack and black gum. In Canada, Norway pine, Douglas fir and tamarack have been used most largely, and with good results. In the Canadian West, Douglas fir and possibly hemlock and tamarack will furnish the supply of timber stick for wood paving, and it is safe to assume that there will be a marked increase in the use of these woods for such purpose. In Eastern Canada, the logical choice of native timber for paving block stock is Norway pine. This species is so well known to members of this association that descriptive comment is unnecessary. However, it may be of interest to note that in structural and physical characteristics this wood resembles rather closely the Baltic pine of Europe, the merits of which for wood paving service have been so conclusively demonstrated in England. Norway pine, as previously noted, is already widely and favorably known as a wood paving block timber, both in the United States and Canada, and the growing recognition of its merits for this purpose will undoubtedly lead to its much more general use in this country. It is not improbable that creosoted hardwood blocks may be adopted to some extent for paving purposes in Canada. At present the limited records of service performance of creosoted hardwood block pavements are hardly sufficiently conclusive to warrant any confident prediction as to developments in this direction.

OLDEST IRRIGATION CONDUIT AND DAM IN THE UNITED STATES.

ON the meadows beside the lower San Diago River, half a dozen miles northeast of the City of San Diago, is the old San Diago mission. Here the Catholic missionaries, working northward from Mexico, established near the end of the eighteenth century one of the first outposts of civilization on the Pacific Coast in what is now United States territory.

In this arid region, with a rainfall averaging only about a dozen inches per annum, all occurring in the fall and winter months, irrigation is a prime necessity to agriculture. There still stand in a remarkably good state of preservation the dam and a large part of the conduit by which water was diverted from the San Diago River and carried some miles down its valley to the ranches surrounding the mission. There is little doubt, according to Engineering News, that this dam and conduit are entitled to the distinction of being the oldest irrigation works in the United States or Canada, with the exception, of course, of the more or less primitive works for irrigation carried on by Indian tribes prior to European settlement. When one bears in mind the extreme limitations in means of transportation, in material, labor, and money with which these Catholic pioneers had to contend, the building of the old Mission dam and conduit was a remarkable achievement.

The dam, it should be explained, is purely a diverting dam and not a storage dam. The San Diago River in its course of some 50 miles descends from elevations of 3,000 to 5,000 ft., and the river and its tributaries have eroded deep canyons in the soft rocks of the region. At its flood stages the river carries a very heavy burden of sand and silt, so that the pond back of any dam built across it is filled full with solid material during the first flood season.

It is of interest in this connection to record that prior to 1850 the San Diago River carried annually vast quantities of silt into San Diago Bay and was rapidly filling up the present harbor of San Diago. It was due to the genius of a distinguished officer of the Corps of Engineers that San Diago harbor was saved for commerce. During the years prior to the Civil War, Lieut. Geo. H. Derby, of the Corps, who was stationed on the Pacific Coast, made himself famous by writing humorous papers which were published under the *nom de plume* of John Phoenix. Lieutenant Derby proved that he was as competent an engineer as a humorist by diverting the course of the San Diago River at its mouth, so that instead of discharging into San Diago harbor it was made to empty into what is known as False Bay, a shallow body of water located to the north of San Diago Bay and separated from it by a high sand ridge.

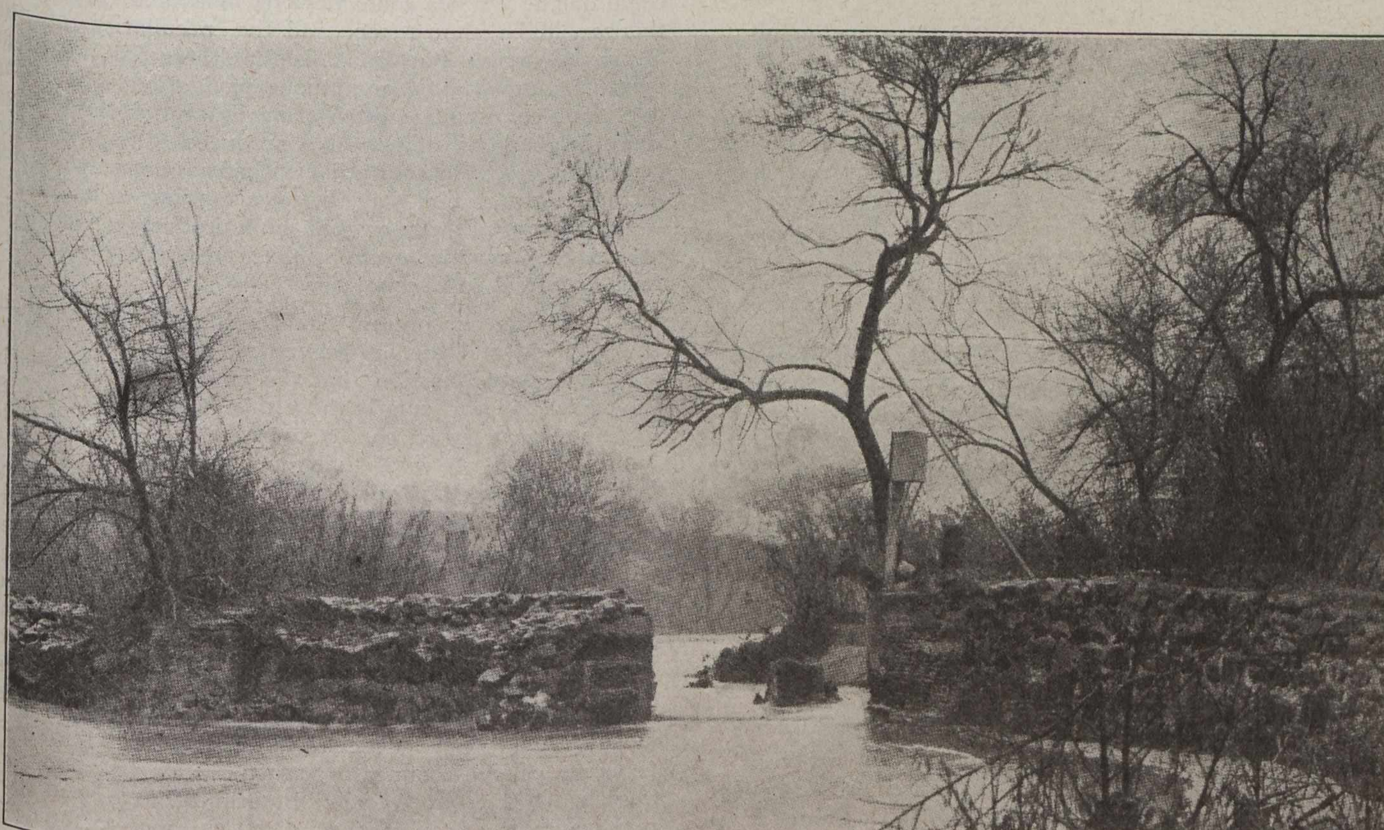

Fig. 1.—The Oldest Irrigation Dam in the United States, Built by the Old Mission of San Diago, Cal.

The old Mission dam and the conduit, as may be seen by the accompanying illustrations, were built of a curious combination of rough rubble masonry and very large, thin tile. The mortar used was, of course, lime mortar and must have been burned in kilns established for the purpose near-by. As may be judged from the views of the dam and of the flood which it annually sustains, this lime-mortar masonry has lasted remarkably well. The central opening in the dam, seen in the view, doubtless originally contained a wooden controlling gate which was left open in time of flood and was closed only when it was desired to divert the water into the irrigation conduit, which connected with the dam near its north end. One reason for the location of the dam at this point is that the river here flows over bedrock at the head of a long canyon, so that all the flow down the valley is intercepted by the dam. In recent years the dam has been used as a river-gauging station by the United States Geological Survey.

From the dam the masonry conduit follows down the north bank of the river about three or four miles to the lands around the old mission. Much of the conduit is still plainly visible from the highway on the opposite bank. The construction of the conduit is shown in the accompanying cross-section drawing. Over a part of its course,

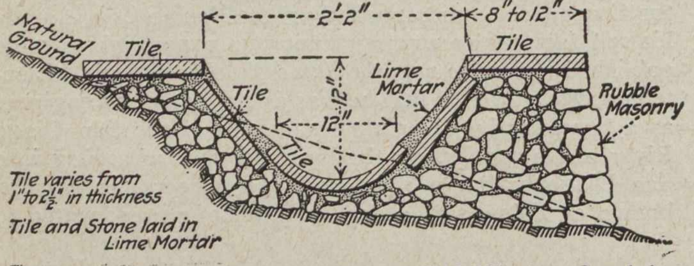

Fig. 2.—Typical Cross-section of Old Mission Conduit Near San Diago, California.

however, the conduit was in excavation and appears to have been merely lined with masonry. The size of the tiles used for lining is particularly noteworthy and evidences the considerable ability of these early pioneers in the art of pottery manufacture and in utilizing Indian labor for this more or less technical work.

DISCUSSION ON ECONOMIC AND STRATEGIC ASPECTS OF ENLARGEMENT OF WELLAND CANAL AND OF CONSTRUCTION OF GEORGIAN BAY SHIP CANAL.

At a general section meeting of the Montreal Branch, Canadian Society of Civil Engineers, Col. W. R. Leonard presented his paper on "Economic and Strategic Aspects of Enlargement of Welland Canal and of Construction of Georgian Bay Ship Canal." Colonel Leonard stated that he had prepared this paper four years ago while in the government service, and that it was not intended as a criticism of any government in Canada.

The paper, a resumé of which appeared in last week's issue of *The Canadian Engineer*, was then read, being illustrated by maps and plans of the Ottawa Ship Canal which, the lecturer stated, were not intended to boom that work in any way.

Written discussions were read, one by J. S. Armstrong, who referred to the haste with which the paper was to be dealt with—which haste has been overcome by the postponement of the reading of the paper for one week.

A written discussion by Mr. F. H. K. Wicksteed, of Toronto, then was read. Mr. Wicksteed stated that water transportation was not always cheaper than rail, and only in the case of our great inland seas could it compete with the railroads. Canal routes, he stated, were not cheaper than rail. Mr. Wicksteed said that there is no basis for the fear that the enlarged Welland Canal will carry a preponderance of United States freighters into Lake Ontario, giving them the control, because they will make more money running from Lake Superior ports to Lake Erie ports than if they went to Oswego. Regarding the Georgian Bay Ship Canal, he stated that a certain gentleman in Detroit said that the canal was not thought of as anything but a joke. They would not send their 600-ft. steel freighters through the Georgian Bay Canal because they would lose money.

Discussion then followed as to relative freight rates via rail and water over various routes.

He stated in reply to Col. Leonard's query as to whether the United States would bear their share of enlarging the St. Lawrence system to divert the trade from Troy and New York to Montreal, that he thought it was only a matter of approaching the United States government and asking for a convention or to appoint a joint commission. He was certain that they would ask for one-half the water power, as called for in the International Waterways Treaty.

Mr. Wicksteed did not agree with Mr. Leonard's statement that the deepened Welland-St. Lawrence Canal would be found to have probably three times the length of actual excavated canal and about the same length of restricted river navigation as compared with the Ottawa route. He said the figures for the excavated channel and canal on the Ottawa route were 94 miles and three times this would be almost 300, and the peculiarity of the Ottawa-Georgian Bay Canal is that there is no great length of unrestricted continuous navigation in it. The maximum continuous stretch of navigation on the Ottawa route is about 32 miles. He produced a map showing these features. Another point is that no matter what our inland transportation may be, we are going to be limited in the amount of grain or other bulk products we can ship via the St. Lawrence route by the transportation facilities from Montreal to Europe.

So far as our present canal systems are concerned, the Welland and the St. Lawrence, it has been demonstrated beyond question that the boats that are now in the trade, and particularly those that have been built with a view to that traffic, can carry grain from Lake Erie to Montreal at lower rates than the railways can carry it from Georgian Bay ports to Montreal. By the time the cars are loaded at Georgian Bay and made up into trains there is a good deal of cost involved, but when they reach Montreal they go into the terminal here where they have to be broken up and the freight carried down to meet the steamship and then the empty cars brought back and made up into trains and the terminal expense is very high. They will always be handicapped in that way as against the steamship which can float right alongside of another steamship and transfer its cargo. Therefore, the greatest economy can be attained in transportation by employing vessels of, say, ten to twelve thousand tons on the lakes and transferring the cargoes at the entrance to the canal to the types of vessels adapted to go through the canal, and again unload and transferring the cargo to ocean-going ships. It will never be possible to build a type of ship that will be economical to run from the head of our lakes to British ports.

Sir John Kennedy stated that it was the first time he had heard the paper and was not prepared to discuss it. However, he agreed with one point in Col. Leonard's paper and that was that the whole question should be thoroughly studied by a commission.

Mr. White quoted some comparative rates for freight via rail and water which Mr. Jamieson said were rather misleading, as they did not take terminal costs into account.

Mr. Leonard made the statement that his paper was written so as to get the Canadian Society of Engineers to look upon these questions and the expenditure of public money as part of their business and to discuss the matter in their own Council.

Col. Leonard was accorded a hearty vote of thanks for his paper.

COST-KEEPING AND EFFICIENCY IN ENGINEERING.

In modern engineering construction the question of how much a dollar will do is one of vital importance. The necessity of cost-keeping enters as largely into general engineering and contracting as into industrial business. Shrewd judgment, rule-of-thumb or the knack of "coming out on the right side" are not the factors in success that they may have been in the past. Costs are as important to the engineer as to the manufacturer.

No one cost system can be designed that is of universal applicability—each engineering task has its own individual peculiarities, and demands an independent treatment and natural development. The success of a system depends to a large extent upon the executive whose brain and administrative ability must plan and put into execution methods of reducing cost: the cost records are merely a guide post on his path. In the installation of a system, time and persistence are important elements; while its successful operation requires loyal co-operation from all subordinates.

The following useful discussion of the fundamental principles are requisites of cost-keeping as applied to engineering construction, is from a lengthy paper on the subject in Vol. viii., No. 37, of "Professional Memoirs" of the U.S. Army, by Lieut S. C. Godfrey, Corps of Engineers.

Tredgold's definition of engineering, "the art of directing the great sources of power in Nature for the use and convenience of Man," has been so often quoted as to become classic; but to-day it is no longer permitted to pass unchallenged. Wellington came much nearer the revised conception of engineering, when he spoke of it, in his "Economic Theory of the Location of Railways," as "the art of doing well with one dollar that which any bungler can do with two after a fashion." The great engineering works of antiquity, many of which were of monumental character, like the pyramids, were built with scarcely any regard for cost. The substantial highways and aqueducts of the later Roman days expressed more of the spirit of commercialism. To-day, more than ever before, considerations of economics play a vital part in every constructive enterprise. The engineer finds himself ever in closer touch with industrialism. One day his scientific knowledge may be applied to the appraisal of public utilities; the next, he is asked to decide the advisability of an investment, or the type and magnitude of a contemplated structure. Again, he is chosen by a municipality, like Dayton, to act as its city manager. In short, he is constantly compelled to go beyond his formulas for stresses in order to answer the question, "Will it pay?"

Efficiency in engineering construction may be defined as accomplishing a given task in the most expeditious, most economical, and most substantial manner practicable. This does not mean, of course, that the cheapest practicable structure is sought. A Gatun dam may be designed with such emphasis on security, with such large factors of safety, as to make its cost necessarily high; but the dam, once designed, is built with the effort to secure, by means of efficient methods of construction, the maximum return for each dollar expended. Now, efficiency is no new invention; but modern industrial conditions have made efficient methods well-nigh a requisite for success. With the increasing discussion and amplification of this subject, the term "scientific management" has become a watchword, and "efficiency engineer" a term with which many have hoped to conjure. Under the guidance of elaborate systems and detailed rules worked out by Mr. Taylor and other experts, shop management has become highly standardized. The application of these principles to field engineering, where the work is of a very different and less specialized character, has been far less widely made; yet much has been done in this direction as well. Cost-keeping systems have been put into successful operation; and although such systems have been far from proving universally successful, their failure, where they have failed, has been due to some weakness in the system or the man operating it, rather than in the principle involved.

Principles of Cost-keeping.—The essence of the question of costs is extremely simple. Man is in business for profit. The margin between the cost of the product he makes or builds and its selling price is the measure of his success. Thus cost is fundamental; and the successful business man, whether he be a manufacturer selling automobiles, or a contractor building a dam, has been the man who, prompted perhaps merely by business instinct and shrewdness, watched and estimated his costs closely. Now, a good cost system should place in the producer's hands a tool far more powerful than any rule-of-thumb methods; for it replaces his guesses by exact knowledge, points out leaks and wastes, establishes standards for comparison, and gives data for future plans and estimates.

This raises at once the question as to what constitutes a good system of cost-keeping. The requisites of such a system may be summed up by saying that the records should be: (1) Reliable; (2) Simple; (3) Immediate.

Reliability.—This is the first essential. If it is not fulfilled, the records may be misleading, in which case they are worse than useless. For this reason all sources of cost must be weighed,—so-called "cost data" are often misleading because they involve simply field cost, without consideration of such factors as depreciation of plant. Figures must be carefully checked and proved, estimates of work to be done compared with measurements of actual performances, the whole tied into, and checked by, the general accounting scheme. The impossibility of mathematical accuracy, on any extensive work, is apparent, when the factors of overhead expense are considered. It is a question rather of practical accuracy, and this is by no means out of reach.

Simplicity.—The cost system should not be too complicated—a requirement which many systems, as designed, fail to meet. The system must be installed, be it remembered, in most cases, in the face of inertia and perhaps opposition on the part of subordinates who are adherents of a system, or lack of system, already in operation; and must be operated, perhaps, by these very subordinates. Then, too, the labor and cost of the system itself, with its considerable clerical work, must be carefully considered with reference to the results it is expected to attain. In particular, the reports should not demand much clerical work from men in the field, foremen and others, to whom such work is distasteful and whose efficiency may be impaired thereby.

Immediateness.—In order to be of practical value, the cost records must be available before the information they portray is cold, while there is still time to act on it. Of little use, at the end of a season, to put one's finger on a leak that should have been stopped way back in midsummer. Yesterday's mistake should be found out in time to plan to-morrow's work. This requirement necessitates that the records be kept up-to-date and frequently posted at stated periods, so that their information will be promptly available. Here appears an additional argument for simplicity, for the more simple the system the easier it should be to obtain prompt returns.

The Test of Usefulness.—If the cost-records meet these three requirements, they will be pretty sure to be used and not neatly filed away on a shelf to gather dust. They will present in concise form just the information the executive needs to keep him in touch with the work. They should be well arranged and summarized, with such standards for comparison as "average cost to date," for example. Of course, the reports should show unit costs, and these should be accompanied by such descriptions of the particular circumstances of the work—climate, distance from railroad, labor conditions, etc., as will make them of permanent and not merely temporary value. The cost system dovetails into the methods of purchasing, the functions of inspectors and their reports, the system of administration. In short, it should present an intimate and ever-changing view, a moving picture, as it were, of the entire project.

Units.—Closely allied to the requirement of accuracy is the question of the choice of proper units in which to express the cost-records. Care and uniformity in regard to their use are essential. For instance, the standard unit for concrete is the cubic yard; but concrete sidewalks are usually expressed in terms of square feet, and so any comparison or transformation from one unit to another is misleading unless it involves the thickness of the sidewalk. Round piles may be described in terms of linear feet, while at the same time sheet piles are called for in feet board measure. Units of length, commonly applied to drilling, pipe laying, etc., and units of area, used in painting, paving and similar work, are in general easily chosen. Volumes are much more difficult to determine with any degree of precision; recourse must be had, in the absence of careful surveys, to estimates of performance, to such records as the number of batches of concrete mixed, and to discrepancies in the stock-pile showings, etc. Measurements of weight are frequently the most suitable, as for steel and dynamite. Other units involve a combination of the simple terms; the ton-mile as applied to transportation and the man-hour in connection with labor are prominent examples. In measuring economic efficiency of performance, the dollar must be used with caution as the unit of comparison. Prices for raw supplies, for labor, fluctuate widely; conditions vary infinitely; and cost-records can be considered of comparative and permanent worth only when they are expressed in such units as to eliminate or at least explain these fluctuating factors.

Classifications.—In devising a system of records for any elaborate engineering work, the need of classification is strongly felt. Thus the different work-operations, the raw materials, the finished products, the workmen themselves, may any and all be subjected to this systematizing process. Not only are a better organization and grasp of detail made possible thereby, but in the records themselves, by the use of proper symbols, economy of time and space may be gained. The systems most commonly adopted are modifications of the familiar decimal classifications of Melvil Dewey's; the use of numbers is sometimes supplemented by that of letters. The whole subject is a fascinating one; such classifications, however, should be designed and applied cautiously. A too complicated and cumbersome classification defeats the very ends it aims for; in the field, especially, there is danger of too much elaboration. The fundamental requirement of simplicity must be kept in mind.

Forms.—In designing forms and record blanks, again, the first requisite is that they be simple and readily understood. It must be remembered that they are but a means to an end, and must not be allowed to become a fetish. As one writer has tersely put it, "About half the blanks and forms of the ordinary business system are merely confetti whereby those who are a lap behind can follow the trail of the live ones." Forms should also be made out with reference to the man who has to fill them out; a foreman will take infinitely more pains and interest in making out a report if it obviously leads somewhere, if it has a purpose that he can understand.

Cost-keeping vs. Bookkeeping.—It has been said above that the cost system should be tied into the general accounting scheme. The question arises, "how does cost-keeping differ, if at all, from bookkeeping?" The answers have ranged all the way from the statement that cost-keeping is merely "intelligent bookkeeping" to the assertion that "cost-keeping should not be allowed to interfere in any way with the bookkeeper's work." Somewhere between these two extremes lies the true relation. The bookkeeper's figures should, perhaps, tell the whole story. His tendency, however, is to confine himself to debits and credits, to the balancing of accounts with mathematical precision. The engineer uses the same figures, but uses them to lay bare the engineering aspects of the job. He is interested rather in quantities of materials, in unit costs, in efficiencies of performance, and must devise the means of obtaining such data. Yet these records of his must dovetail into, and be checked by, the accountant's figures. Thus the two sets of records are complementary; and to design an adequate system of cost-keeping requires a familiarity with the principles of accounting, plus a comprehensive knowledge of the engineering features of the particular task involved.

Factors of Expense.—All factors of cost will here be grouped and discussed under the following heads: (1) Labor; (2) Materials and Supplies; (3) Equipment; (4) Overhead Expenses.

Labor.—(By the term labor is here meant simply the direct or productive labor that can be clearly charged to one activity.) The fundamental records that pertain to labor costs are the time record and the pay-roll. The latter needs no particular comment here. The time-records should show not only the total time of work of each employee, but also the division of that time as desired, on different activities. As far as practicable, these records should be certified, either by the employee himself, as is feasible for an office force, or by foremen or time-keepers, in the case of laborers; for the signature adds materially to the reliability of the record. As to the relative value of foremen's and time-keepers' reports, opinion differs. Some writers argue that the former, with the certification of the foreman to the time-record of each man in his gang, are much more accurate and valuable than the report of a time-keeper who can be with that gang, at best, but a small part of the time. Other engineers, and more especially men of practical experience, point out that a field foreman, as a rule, cordially dislikes to putter with clerical work, and that both the report he is compelled to make out and his work of supervision are likely to suffer in consequence.

Again, the question arises as to the relative value of written time-reports, and those punched on prepared forms. The former are more flexible, require rather less explanation, are more easily erased. The latter are more uniformly clear and legible, should be more quickly made out, convey information in a more condensed form, provide an extremely simple means of duplication, and can be neither smudged nor fudged.

But whether time-cards or foremen's reports be adopted, whether the reports be written or punched on a printed form,—these are questions which must be decided in every case upon consideration of the particular job and workmen involved.

Materials and Supplies.—These two terms are usually differentiated, the former being applied to raw material that becomes a part of a finished product, like cement, steel reinforcement, etc., the latter being used to designate auxiliaries necessary to carry on the work, which are used up in course of production, such as coal, oil, small tools, etc. These classes together form an element of the cost-keeping system that must be carefully considered. Inefficient handling and waste of material are usually responsible for a large share of excessive costs, and it is only by an efficient stores system that these wastes can be minimized, and the costs properly apportioned among the various activities. Such a system involves, besides the purchasing division of the office, one or more storekeepers, who care for this property, issue it to the field upon proper requisition, and receive receipts for all articles issued. At the end of the month these receipts are tallied with the inventory and "stores-received" accounts. The best form of inventory is some variation of the so-called "perpetual inventory"; by its use a record of the amount of each class of material at hand is always available, and the stock-taking need not be concentrated into a brief frantic interval, at the end of the year, but can be done gradually, an item at a time, the ledger or card-system being always posted for comparison. Such an inventory, to be sure, requires "perpetual" attention to keep it posted to date.

Some such system as the one outlined above is necessary in order to form a guide for purchasing, prevent undue wastes in handling, and provide for even an approximately accurate distribution of costs. Certain classes of material, of course, will go to the job in an entirely different and perhaps more direct way,—the sand for concrete dredged from a river-bottom, for instance. Their part in the cost-keeping system requires no special comment here; except that whatever manner of handling the materials be adopted, such records of quantities, labor, etc., should be so kept as to disclose any excessive waste or unnecessary expense.

Equipment.—This term includes all costs directly connected with units of plant, tools, etc., which have a more permanent status than the "materials and supplies" already considered. The proper care and preservation of such property is an important factor in minimizing costs, and the cost-records must consider the first cost, life, and usefulness of each important unit of equipment. The problem of allocating the equipment costs to the proper sub-divisions of a job is manifestly a complex one, in view of the distribution of the services performed by the plant. One solution often adopted is to charge each activity a (book) rental for the services of any unit of equipment, while employed in that capacity. This rental is an arbitrary rate based on the factors: first cost of the unit, interest on the investment, operating charges, maintenance and repairs, time lost through idleness, and depreciation.

Another method of handling equipment costs is to charge directly to work benefited such items of plant operation as can be fairly placed against it. For instance, if a dredge be placing fill in a cofferdam, the daily labor cost of operation, ordinary repairs, and the daily supplies of coal and oil, can fairly be charged directly to "cofferdam fill." On the other hand, such charges as those for more extensive repairs, and time lost through idleness, can hardly be distributed oftener than on a monthly basis; while the question of depreciation of plant, and interest on the investment, will probably be considered annually.

Of these two methods, the first seems to be generally preferable, in that it is on the whole simpler, and contributes immediately to give a total cost which is at least an approximation of the true total. The rental charge, to be sure, will never balance exactly with the true cost of the service, but must be revised from time to time, and the discrepancy adjusted.

Of the terms used above, the word "depreciation" requires some special comment; the application of all the others is sufficiently clear.

Depreciation is the lessening in value of any perishable property. The neglect to take into account this factor of cost, intangible though it is, leads to cost showings that are not justified by facts. The application of depreciation to certain engineering problems, such as the evaluation of the railroads, may raise difficult and complex questions. For the present purposes, however, it suffices merely to describe the most practical working means of accounting for the depreciation of equipment engaged in engineering work.

The Straight-Line Method.—This regards the total depreciation (first cost minus remaining value to end of probable life) as divided equally over that entire life period, so that the annual depreciation to be charged off is a constant. In applying this method, of course, the remaining value and probable life in each individual case, must be estimated with the help of any available data based on actual experience. This is the simplest and most general method of calculating depreciation for our purposes. A rough general rule recommended by some writers is: annual depreciation on machinery, 10 per cent.; on wooden buildings, 5 per cent.; on brick buildings, 3 per cent.; but such approximate generalizations have little value in this connection.

The Sinking Fund Method.—In this case it is assumed that a fixed annuity is to be set aside at interest to form a sinking fund, which, at the end of the probable life, will amount to the replacement value of the unit of plant. The depreciation at any time, then, being equal to the accumulated theoretical fund, will not increase uniformly, but slowly at first and more rapidly later than by the straight-line method. For certain purposes this method, which is based on the law of compound interest, is better adapted than the first.

Still other methods of treating this question have been devised. Their requirements are not necessary, however, for the simple case under consideration. What is wanted is a working formula, such as that furnished by the straight-line method. While annual depreciation has been referred to, implying a yearly consideration of this question only, it may be advisable to figure, or at least charge, the depreciation every month.

Overhead Expenses.—This term includes a great variety of indirect costs that are not covered in the foregoing classes. Chief among these are: supervision and miscellaneous labor costs; office expenses (rent, telephone and telegraph, etc.); heat, light and power; interest on investment; accident and fire insurance; taxes. The element of "plant," with its upkeep and depreciation, which is sometimes included under this general head, has already been discussed separately. The application of the cost-records to this class of expenditures is perhaps the most troublesome of all, so difficult is it to find an accurate method of prorating these charges. In general, it can be said that the distribution of costs should be made so that each activity will be charged in proportion as it has been benefited. But on just what basis the division is to be made,—whether in proportion to the total expenditure in each department, or in proportion to the direct labor costs, or by whatever other method,—all this must be decided upon consideration of the special circumstances in each particular case.

It is recognized that some of these factors of cost merely mentioned above,—depreciation, interest on investment, etc.,—frequently have an important bearing on the larger aspects of the project considered as an investment. These, however, do not ordinarily lie within the province of the engineer in charge of construction, and are somewhat outside the scope of this article.

WHY HOUSE CONNECTIONS TO THE SEWER SHOULD BE CAREFULLY MADE.

SEVERAL years ago it was the general practice to use a sewer system to carry off any water or drainage which was objectionable to the property owners. For this reason roof leaders and cellar drains have been connected to the sewers, and according to C. G. Wigley, C.E., in "The Cornell Civil Engineer," most of the connections have been made with poor jointing material, or none at all, so that ground water would be admitted to the sewer system. This was all very well as long as the drainage or sewage could be discharged into the sewer system, and then required no further care or consideration. In those days the plumber, or contractor, was permitted great latitude in the method of laying the connections to the sewer, and also in the manner of making the joints in the sewer system, or house connections. The pipes used were sometimes of that class which to-day is known as seconds or culls. The joints were sometimes made with cloth wrapped about the pipe, with clay, or with inferior qualities of cement mortar, the method used depending largely upon the particular ideas of the person doing the work.

In those days the combined sewer system was the type in use as a general rule. The general idea was to discharge the sewage and drainage water from the surface of the ground, and certain portions of the sub-soil drainage into the sewer system in the least expensive and easiest manner. There were very few people who thought that these wastes after being discharged into the sewer system would ever require any further consideration. The old idea, which was fallacious in many respects, that a stream was self-purifying, was deeply rooted, and it required several years of nuisances on some streams before the limitations of the self-purifying actions of the stream were investigated and determined. At about the same time investigations showed that sewage-polluted streams were not safe sources of drinking water, even though the offensive odors and nuisances were not apparent at the point at which the water for human consumption was taken from the stream. These considerations, with others of equal importance, resulted in the enactment of laws in several of the more densely populated states, necessitating the construction of sewage disposal plants in order to properly dispose of the sewage.

This action led to a change in policy as to the methods of constructing sewer systems. Obviously a greater quantity of sewage required larger treatment works. Therefore, the separate, or sanitary sewer system, then came into greater use. In this system of sewerage it was intended that only the grossly offensive house sewage was to be received. No provision is made for roof drainage, surface drainage, and only a limited allowance for sub-soil drainage. It was soon found that it was not an easy matter to keep underground water from entering the sewers through leaky joints. The amount of water entering the sewers in this manner was so great, as in some cases to make the sewage disposal plant inoperative for several days after each heavy rain storm. This is a serious matter where the sewage plant is located above some water supply used by a neighboring city or town. Attempts have been made to prevent ground water from entering the sewer system by constructing underdrains, and by improving the methods of making the joints. The underdrains are often jointed tile covered with broken stone, or gravel, and laid alongside and below, or directly beneath the main sewer. These drains receive the ground water, and discharge it into nearby water-courses, or storm water conduits, and thus prevent the ground water from rising above the main sewer and thus entering leaky joints. The method is only partly successful.

In order to make the joints in better manner various materials of a compact and waterproof character have been tried. These compounds include mixtures of sulphur and sand; mixtures of tar and oil, with other ingredients, making a waterproof joint material, such as Jointite and G. K. Compound. These and some other compounds are poured while hot, thus materially increasing the cost of making the joints. The use of these compounds has in many cases reduced to a minimum the infiltration of ground water in the main sewers, and tests made on sewer systems, just completed, where such materials are used for jointing often give surprisingly low figures as to leakage per mile of sewer. These tests have been made before the house connections were laid. In several instances it was noticed that as soon as the house connections were made, the amount of ground water entering the sewers after a rain storm increased at an alarming rate. This leakage, or infiltration, in some cases was so great that the pumps and disposal works provided were entirely inadequate to properly care for the flow of sewage. At one place, for instance, the sewage treatment works provided were designed to care for a maximum wet-weather flow of 900,000 gallons per day. This figure included an allowance for probable ground water infiltration. Imagine, however, what the result must be when over 2,000,000 gallons of sewage and ground water is at times discharged at the disposal works. At another place the wet-weather flow is so great that the sewage after heavy storms flows out of the top of the manholes. At still another place the sewage was backed up into cellars and flowed out of the manholes into the streets because the unexpected volume of sewage was more than the sewage pumps could take care of. Investigations made at various places failed to show that any appreciable amount of this water came from roof leaders or surface drains secretly connected to the sewer system. In order to give some idea of the direct financial loss incurred by the admission of this ground water, the following instance is particularly mentioned: At one plant situated on a small inland stream, where the sewage from a population of about 6,000 persons is treated, the wet-weather conditions were so bad that it was considered advisable to institute some form of remedial measures in order to protect a large water supply, taken from the stream some distance below the town in question. The estimated cost of the necessary changes was $35,000, or $5.83 per person. This is a rather heavy tax.

The investigations mentioned above indicated that most of the leakage was due to the poor methods used in making the house connections. In one instance the contractor was found using clay for the jointing material on the house connection pipe line. In defense of the use of the material he stated that, "All the other connections in town were made in the same manner." The use of clay as a joint material was as early as 1878 discussed by a leading sanitary engineer, as follows: "The material most commonly used for jointing pipes is clay, which is one of the worst materials that could be found for the purpose." There is no reason for any change in this opinion expressed so long ago. It was also reported to the investigators that joints on the house connections had been made with mud. This on inspection proved to be a very weak mortar made with a dirty sand of poor quality. Instead of the clean sharp sand called for in the town ordinance, a dirty loamy sand had been used. Instead of a mortar, consisting of one part cement and two parts sand, the mixture was about one part cement to six parts sand. It was found that in some cases in removing the caps from the Y-branches that the latter had been broken and then had been patched in very crude ways, so that large quantities of ground water were admitted to the sewers. In one town the wet-weather flow from leaky house connections was considerably reduced by requiring that in wet trenches the connections should be made with cast iron pipe with lead joints. It may be thought that a few leaky joints or a broken pipe on a house connection is not a serious matter, until a person realizes that the total length of the house connections is generally equal in length to the main sewer system, and in some cases even double the length of the main sewer. For example, if a main sewer has a connection every 25 feet, and each of these connections is 25 feet long from the main to the house, the total length of house connections is equal to the length of the sewer. Consequently, a few leaky joints on each sewer connection are in wet soils often capable of overloading the main sewer.

Some people believe that the admission of ground water into a sewer system is desirable. This belief is based upon the assumption that it is necessary to wash out the sewers, and provide a flow of water adequate to float the solid material. The benefits of this practice, however, are greatly overestimated, as a properly constructed sewer on a proper grade will cleanse itself, and if it is necessary to flush sewers laid on a low grade, it is better to rely on some means of flushing that can be readily controlled, such as the periodic flushing of the sewer with a fire hose, hand-operated flushing manholes, or automatic flush tanks.

Apparently the advisable methods of preventing the infiltration of ground water into the sewer system by way of the house connections is to improve upon the methods of making the joints, and also to improve upon the methods of placing the caps in the Y-branches. For the latter purpose disks of galvanized iron have been used, but would appear to be of little value, due to the rusting out of this type of cap in unused branches. The ordinary terra cotta caps may be held in place with a gasket of oakum, or jute, completely filling the space between the sides of the cap, and the bell of the Y, and a thin rim of cement placed over the oakum to hold the cap in place.

As to the methods of laying the pipe, some of the patented jointing compounds would be of great advantage when used with care, and by using pipe 3 feet in length only two-thirds as many joints would have to be made. The objections to some of the present methods of making house connections have been pointed out above, and the financial aspects of the situation have been briefly outlined, hoping that this subject will be discussed, and that rational methods for the prevention of this costly infiltration may be advanced.

In closing, it is desirable to indicate that there are certain sewer systems where the admission of certain portions of the ground water is permissible, but this privilege should only adhere to the combined system of sewerage, and even in this case the privilege should not be abused.

HIGHWAYS.

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In the first article of this series the general principles governing the construction and maintenance of public roads was discussed, and a classification into Country Roads, Branch and Main Highways was suggested. The second article dealt with the first of these, and it is proposed in this article to consider Highways.

It is to be understood that the distinction made is between roads constructed of material in place and those whose roadbed, or at least the centre portion of it, is built up of material brought and placed therein. This improved centre portion varies in width from 10 to 24 feet, a width of from 12 to 16 feet being all that is required in most instances. The object of bringing in material and placing it on the natural soil is to make a hard, smooth surface upon which the rolling resistance will be small, due to its not yielding under the wheel loads. To accomplish this it is necessary that the natural soil, which must ultimately carry the load, be not exposed to too great an intensity of pressure. The filling material is compacted together and has more or less rigidity. The wheel loads cause a certain pressure over the surface of the road in contact with the rim, and this pressure is distributed by the filler to a much larger area of the soil supporting the roadbed. The principle is the same as that used in house foundation, bridge piers, etc.

The filling material is placed on the natural soil, which has been previously thoroughly compacted by rolling. This subgrade, as it is termed, may be either flat, requiring a greater depth of filling material at the centre to form the crown, or may conform to the shape of the finished surface, giving a uniform depth to the filling material throughout. The amount of material required depends upon the heaviness of the traffic and on the method of maintenance. There is a constant wearing away of even this hard roadbed, and this may be replaced either periodically or by continuous upkeep. In the former case the initial depth must be such that the improved surface will not wear too thin. This minimum allowable thickness depends on the traffic and the nature of the subgrade. The greater the depth, the larger is the area of subgrade over which any wheel load is distributed.

The filling material must be sufficiently hard to resist the crushing action of the loads and sufficiently tough to prevent its breaking under the continual blows to which it is subjected by the traffic. There should also be a binding together of the surface coat and of the body of the roadbed in order that water falling on the road shall not find its way down to soften the subgrade. The materials generally used are gravel and broken stone, but if neither of these is available any material which would satisfy the requirements might be used as a substitute. One of the functions of the alert highway engineer is to utilize local material, such as well-burned clinker, hard brick spalls, slag or other by-products, which may be economically used in the body of the roadbed, and thus save the expensive imported material for the wearing surface.

Pit-run gravel should not contain too great a proportion of sand or of clay if satisfactory results are to be obtained. About ten per cent. of clay or loam is necessary to act as a filler and binder, but more than this is apt to wash to the surface and cause a muddy and dusty road. It is not possible to get as compact or waterproof a road with gravel as with stone, and hence it is more suitable for the lighter types of traffic.

There are two types of construction in broken stone roads, i.e., macadam and telford, named after the English highway engineers who first used them. The difference lies in the fact that in a macadam road the layers of small stone (up to ¾-inch in size) forming the filling material are laid directly on the subgrade, whilst in a telford road a layer of large stones is first laid on the subgrade and the smaller stones laid on these. Thus, when we speak of a macadamized road we mean one in which a 16-foot strip (more or less) in the centre is filled 6 in. to 8 in. deep with small, broken stones laid in layers on a rolled subgrade, each layer rolled and compacted, and the whole held together with the pulverized dust, which, when mixed with water, forms a cementary paste.

Not all stone is suitable for this purpose. Hardness, toughness and a cementary action on the part of the dust is necessary. It is possible, experimentally, to determine the characteristic of any proposed stone, and this should be done before any great outlay is undertaken. In fact, it might be safely said that a properly operated experimental department is an essential to any broad scheme of roadway improvement.

The comparative tractive efficiency of a hard road with a smooth surface and an unyielding roadbed is shown by the study of the following loads, which could be hauled at two and a half miles per hour on the level by a 3,000-pound team drawing an ordinary well-lubricated farm wagon with 2-inch tires: Brick pavement, eleven tons; asphalt pavement, six tons; macadam road, five tons; gravel road, four tons; ordinary earth road, two and a half tons; ordinary sand road, two tons. In using such comparisons in estimating purely financial benefits full loads only should be calculated, and the necessary cost in grade reduction and bridge construction to allow of heavier loads on the improved roadway must be allowed for. As has already been pointed out in the first article, the cost per ton mile to a farmer and a freighter are not the same, and deductions which apply to the latter cannot be used in their entirety in calculating the saving to the former. The ease, speed and frequency of intercommunication with neighbors and with local centres, which does so much to ameliorate agricultural conditions of life, should be kept prominently to the forefront when discussing road improvements.

The cost of both gravel and broken stone roads is comparatively large. Local conditions affect the cost materially, but for purposes of general comparison with earth roads it might be said that where material can be obtained locally the cost of a gravel road would be from $1,000 to $2,000 and the cost of a broken stone road $4,000 to $6,000 per mile. The cost of upkeep for a gravel road might be placed at $75 and for a broken stone road at $200 per mile per year. The problem which has to be faced in Alberta is that over the greater portion of the Province stone and gravel are conspicuous by their absence, and there would have to be a large additional cost to cover freight charges. Just to illustrate this, the cost for gravel alone at $1 per cubic yard for a road 16 feet wide, with a 1-foot depth of gravel, would be over $30,000 per mile. It would seem, therefore, that there exists an excellent opportunity for experimental work looking toward the utilization of other available material.

As has already been suggested in regard to earth roads, a judicious mixture of sand and clay might solve to some extent the difficulties met with in so many places where the material in place is either sand or clay, neither of which makes satisfactory roads. The clay expands with moisture, becomes very soft and plastic, and when dry contracts, leaving cracks in the roadbed. If rutted, the dried clay ridges are difficult to work down again into a smooth surface. Sand, on the other hand, expands and runs easily when dry and contracts when wet. A judicious mixture of the two would counteract the adverse tendencies of each, and so make a roadbed with a more permanently compact body which would withstand the action of the rain, be easier to work into proper shape when rutted, and possess a surface giving a better grip than does a clay road in wet weather.

In conclusion, it might be said that much remains to be done along the lines of improving existing earth roads and experimenting with mixtures of material locally available. A great improvement can be brought about by so simple a thing as systematic dragging. It will be some time before we are justified in seriously undertaking the construction of roads equivalent to the macadamized highways of older countries, where rock is more plentifully distributed than it is in Alberta.

MEASURING THE DISCHARGE THROUGH A VENTURI TUBE WITH A DIRECT READING METER.

A new method has been described by J. Dejust in Comptes Rendus, in which the usual apparatus for measuring the pressure difference between the full and the contracted sections of the tube and the integrating mechanism are replaced with an ordinary meter. The difference in pressure between the large and small sections of the tube causes a discharge between these two points.

The intake of the meter is directly connected to the full up-stream section of the tube; between the outlet and the down-stream contracted section of the tube a diaphragm is introduced. The algebraic expression of the ratio of the discharge through the tube to that through the by-pass containing the meter shows that this ratio is not constant. By adjusting the section of the by-pass tube and its diaphragm, however, the terms which are not constant can be reduced to a negligible value and the ratio made substantially constant for various rates of discharge. The constancy of this ratio has been tested on a line 100 mm. in diameter, provided with a Venturi tube having a contraction of 16 to 1, with a by-pass 20 mm. in diameter, a meter orifice of 12 mm., and a diaphragm calculated to make the ratio of meter discharge to flow through the line, 1 to 100. By varying the velocity in the line from 0.16 meter to 1.23 meters, the ratio varied from 103.82 to 105.53; that is to say, 1.65 per cent. of the smaller number. Reckoning from the mean of these two values, 104.675, the maximum error will be 0.82 per cent. of the true discharge.

Statistics show that since 1908 the increase in the use of creosoted wood block in the United States has been very rapid. For example, in 1908, 1,260,000 cubic feet are reported to have been laid, which amount was increased to a total of 10,000,000 cubic feet in 1911. Recent years have shown even greater increases, 1914 alone approximating 4,800,000 cubic feet of wood pavement.

CONCRETE HIGHWAYS SUBJECTED TO EXTREMES OF TEMPERATURE.

By H. S. Van Scoyoc, Assoc.M.Can.Soc.C.E.*

THIS paper is based on observations made by the writer in every province of Canada, except British Columbia. Pavements under construction have been observed, and in some cases have been inspected year after year with a knowledge of materials and methods used in their construction. Temperature ranges of 120° F. are not unusual and in some cases reach 150° F. In some sections below-zero weather occurs for several weeks at a time, so that the depth of frost is greater than in most parts of the United States. The effects of these conditions on concrete roadways can be discussed in three groups:

(1) A breaking up of the surface due to mechanical frost action.

(2) Expansion and contraction due to temperature changes with the formation of transverse cracks.

(3) The upheaval of slabs or parts of slabs with the likelihood of longitudinal cracks occurring.

Mechanical Frost Action.—While it is possible that a porous concrete, weak in cement, might tend to break up on account of expansion caused by freezing, there is small likelihood of this trouble occurring if a satisfactory quality of concrete is secured. No actual case of this kind has ever come to the writer's attention. This must not be confused with the damage that results when concrete is laid in freezing weather or is frozen before it has had sufficient time to set up properly. This is a real danger and is one difficult to remedy after it has taken place. In addition to rutting or picking up under traffic concrete roadways laid in freezing weather may lack sufficient strength to withstand the stresses that the succeeding winter and spring induce.

Expansion and Contraction Due to Temperature Changes.—The first thought with regard to the effect of such ranges of temperature on expansion and contraction is that it would prove serious. When it is realized, however, that most paving is laid at a temperature of at least 60° F. and that the thermometer will rarely rise above 90° F., the amount of expansion to be provided for on account of temperature change is not large.

The drop in temperature in winter produces a contraction which to a considerable extent provides for expansion during the hot weather. Contraction is very evident during Canadian winters. In 1914, the Canada Cement Company, Limited, laid a concrete road about one-half mile in length at their Point Aux Trembles plant, near Montreal. It was constructed to serve as an outlet for heavy traffic from the plant, but was to some extent an experimental road. About 1,000 feet was one-course work reinforced, a second thousand feet was two-course work reinforced, and the remaining section was standard one-course work without reinforcing. Hydrated lime equal to 10 per cent. of the cement by weight was added to about one-half of each section. In both of the reinforced sections an attempt was made to have the work continuous, special care being taken to have the new concrete bond with the work of the preceding day, the reinforcing in all cases overlapping. For several weeks after the concrete was laid the weather was quite warm and apparently all of the sections had bonded, for no transverse cracks were noticeable, although a close watch was kept from day to day. A sudden change in temperature, of about 50°, occurred, and over night a noticeable transverse crack appeared between successive days' work except in one instance where less than 50 feet of concrete had been laid during a day owing to a mixer break-down. During the winter some of these joints opened to a width of more than half an inch. Other instances could be cited showing very noticeable contraction. At present the writer cannot recall a single instance, however, where there has been creeping of one slab on the other or evidences of buckling where transverse joints ¼ inch wide have been provided at intervals not greater than 35 feet and have been properly maintained.

It is during the winter and very early spring that the contraction is most marked, and there does not seem to be the lag due to moisture change that has been reported from localities further south.

While some transverse cracks are undoubtedly due to improper filling over pipe or box culverts, the writer believes that many of them represent carelessness in stopping work other than at a vertical joint.

Heaving.—Concrete pavements in Canada have shown evidences of heaving. During the winter of 1913 a street at Steelton, Ont., adjoining the Canadian "Soo," was raised by frost two inches from actual levels taken by the town engineer. It returned to place with no damage except a longitudinal crack. In a street in Truro, N.S., laid in 1913, one slab raised about two inches, as was shown by its elevation above the combined curb and gutter adjoining. It settled back into place in the spring without even developing a crack.

Station Street, Oakville, was paved in 1914. In a length of about a mile there is only one spot where cracks have developed and there is known to have been an underground spring there that was not properly taken care of.

This work is reinforced and the crack has not opened to any noticeable extent. It has been given no repairs to date.

An exceptional spell of warm weather in January of this year took all of the frost out of the ground along the Toronto and Hamilton highway. The completed portion, about 17 miles in length, was carefully inspected. In only one spot had longitudinal cracks developed, and there only three slabs were affected. At this particular spot the side ditches were not taken down to their full depth until after the concrete had been laid. In digging the ditches, quicksand was encountered and it is now very evident that when the thaw saturated the ground there was a lateral movement of the subgrade material into the open ditch on the north side of the road, leaving the slab on that side unsupported. It settled and a longitudinal crack developed. The slab is noticeably lower on the north side of the crack than it is on the south side.

During the summers of 1911 and 1912 there was laid near Winnipeg, Man., several miles of what has sometimes been spoken of as a concrete road. It was actually constructed as a base for an asphalt pavement, but in an endeavor to reduce the initial cost it was not covered. The mixture was about 1:3:6, and no transverse joints were made. It was laid on the natural soil, gumbo, a very retentive clay, and was given only the attention that subbase work usually gets. It developed transverse cracks approximately every 30 feet, and during the first winter developed a number of narrow longitudinal cracks. It received no maintenance. The second winter opened up the longitudinal joints until many of them were more than an inch wide. By another spring some of them looked like gullies.

Less than a mile from this location there was laid in 1913 a road under much more satisfactory conditions,

*Chief Engineer, the Toronto and Hamilton Highway Commission.

although it also was laid on the natural soil and the drainage provided was field tile placed only about 10 inches below the surface. This road went through the first winter with only about eight cracks in a length of 2¼ miles. The second winter, however, largely on account of defects in the drainage, developed a much larger number of cracks, although the road is still in a satisfactory condition.

In the late summer of 1912 a portion of the King Edward highway, passing through the village of Napierville, Que., was laid of concrete, being more or less an experiment by the Department of Roads of the province. The grading was carefully done by day labor by the Department of Roads and was thoroughly rolled. The concrete work was carefully done by a conscientious contractor with many years' experience in concrete work. This road has passed through four winters. Four cracks have developed. They have been filled once each season and the road is as good as when built, although it gets all the through traffic from Montreal to New York State, as well as the local traffic of the village.

Other instances could be given of longitudinal cracks developing, but enough has been said to show that in some cases the trouble has been caused by settlement of the sub-grade near the edges of the concrete, due to improper preparation of the sub-grade. Sometimes boggy spots beneath the concrete have not been properly drained; the system of drainage provided has not permanently kept the sub-grade dry. In practically every case mentioned there is sufficient evidence to show that lack of proper care rather than climatic conditions has led to cracks. Reinforcing has had a beneficial result in some cases at least, by adding sufficient tensile strength to avoid the formation of cracks.

Canadian weather does not prevent the building of successful concrete roads, but it does serve to emphasize the advisability of thorough drainage, the careful carrying out of proven methods of construction and the absolute necessity for a maintenance system that takes care of defects when they appear.

A great irrigation system in India was opened officially in December. It comprises three separate but connected canals—the Upper Jhelum, the Upper Chenab and the Lower Bari Doab. These aggregate 322 miles in length, with about 22,645 miles of auxiliary channels and laterals. They provide for the irrigation of about 2,200,000 acres of arid land in the Punjab province, in the northern portion of the country. As described in a paper presented before the Institution of Civil Engineers by Sir John Benton, the eastern portion of the Punjab had a tract of 1,500,000 acres of arid but good land which could not be irrigated from any water supply near at hand, owing to previous utilization and reservation of such supply. On the western side of the province the Jhelum River provided a large and unused amount of water. To deliver it to the arid lands necessitated extensive and difficult works, with the crossing of two large rivers and numerous mountain streams and torrents. Bridges are spaced at average intervals of 1.6, 1.5 and 3.4 miles for the three canals respectively. The regulating works, etc., are largely of brick. Chambers in the floors provide settling basins for the silt and so reduce erosion of the masonry floor. Inspectors' houses are placed at intervals of about 10 miles. There is a complete telegraph system. Flour mills have been located at some of the falls on the canals. The project has been carried out by the provincial government of the Punjab at a cost of about $35,000,000.

COAST TO COAST

Calgary, Alta.—The new concrete pier under the Ogden bridge has been completed.

Toronto, Ont.—The new main sewer on Dundas Street, between Humberside Avenue and Runnymede Road, has been completed.

Brantford, Ont.—Freight services on the Lake Erie & Northern Railway between Brantford and Galt was opened on March 1st, giving Brantford connection with the main line of the C.P.R. at Galt.

Spirit River, Alta.—The Gurney Scale Company, of Hamilton, Ont., recently shipped a 6-ton dump scale to this place. This will be the furthest north in Canada that a dump scale has ever been operated.

Victoria, B.C.—The E. & N. Railway Company has been given until April 10th to sign the agreement respecting the proposed Johnson Street bridge, otherwise no further negotiations will be carried on with it.

Vancouver, B.C.—An arrangement has been made for the Dominion Government to pay the province more than the latter's $300,000 for the Kitsilano Reserve and to hand it over to the harbor board to develop as an industrial centre.

Sarnia, Ont.—At a meeting held to discuss whether it would be advisable to use 25-cycle power from the Hydro or maintain the present 60-cycle plant, it was shown by the Hydro engineer that the former was much cheaper for power purposes.

Ottawa, Ont.—A deputation from the Trent Valley was given encouragement in its application for hydro power for eastern Ontario. The obstacle at present lies in disputed jurisdiction of the provincial and federal governments over water powers.

Ottawa, Ont.—It was announced during the budget debate that a new process for refining nickel had been discovered in Canada by which 100 lbs. of matte could be converted into 50 lbs. of metal in 48 hours. The process will be applicable to low-grade as well as high-grade ores.

Victoria, B.C.—The contract for the Canadian Northern Pacific Railway Company's bridge over the upper arm of the harbor has been let and plans are under way for the permanent bascule span which will provide for a clear opening of 70 ft. for purposes of navigation.

Toronto, Ont.—The Ontario Legislature will approve the plans of the engineers of the Hydro-Electric Power Commission of Ontario for a large power development in the Niagara Peninsula. It is said the immediate development will be 100,000 h.p., and there are visions of an ultimate capacity of 900,000 h.p.

Vancouver, B.C.—It is expected that arrangements will be completed shortly whereby the Canadian Northern Railway will purchase right-of-way along the Fraser River, which will enable it to link up its main line with steel already laid on Lulu Island, extending to the proposed permanent ferry terminus at Woodward's Landing.

Moose Jaw, Sask.—The Canadian Pacific Railway has installed a pumping unit at the high-pressure reservoir. This, in conjunction with the advent of milder weather, has relieved the water situation considerably. More water each day is being secured from Caron, and if the warm weather continues the flow will soon commence to approach normal.

Editorial

SELECTION OF ENGINEER OFFICERS.

At the last annual meeting of the Canadian Society of Civil Engineers, a resolution was framed offering to co-operate with the Dominion Government in the training of competent officers for the engineering branches of the service. The resolution also tried to impress upon the government the importance of requiring that all engineer officers should have had practical engineering training before receiving commissions. This resolution was duly forwarded to Sir Robert Borden and Sir Sam Hughes.

The Council of the Society has not as yet made public the replies that it received, but it is unofficially understood that the replies were of a most unsatisfactory character. If this is true, it is most unfortunate. The Canadian Society of Civil Engineers is in the best possible position to assist the government, not only in the training of engineer officers, but, what is more important still, in the selection of those officers. The military authorities cannot be so well posted regarding the engineering ability of the various applicants, as are the members of Council and the branch executives of the Canadian Society.

The Canadian authorities would be well advised were they to follow the lead of the British authorities in this regard. In England no person is admitted to the Royal Engineers except on certification by the president of the Institution of Civil Engineers. An inspection of the form that must be used by all who apply in England for appointment to a commission in the regular army during the war, shows that a candidate for a cavalry regiment must apply to the officer commanding a cavalry regiment; that a candidate for an infantry regiment must apply to the officer commanding a service battalion or a battalion of the special reserve; that a candidate for the Royal Artillery or for the Army Service Corps must apply to the War Office; and that *a candidate for the Royal Engineers must apply to the president of the Institution of Civil Engineers.*

CIVIC IMPROVEMENT LEAGUE.

Every opportunity offers to the Civic Improvement League to do noteworthy work. If political patronage and personal glorification are not allowed to dictate the actions of this body, some real good can be expected to result from their efforts. We suppose it is only natural that in launching any national movement of this sort, some persons must be included in the organization who have little or no contribution to make excepting nice-sounding speeches.

It will be unfortunate, however, if too many politicians and self-seekers are permitted to mould the affairs of the League. The chairman of the Dominion Council of the League undoubtedly has borne this in mind, however, because the list of provincial and national representatives that he has named appears to be more free from this sort of thing than one might expect.

It is worthy of comment, however, that among the list, as published in the daily newspapers, there does not appear the name of any representative of any engineering organization. One could reasonably suppose that the Canadian Society of Civil Engineers would be represented very strongly in this movement. Engineers should take, and should be permitted to take, not only a large part but the largest part in this movement, because, by their training, engineers are most fitted to deal with questions of civic improvement. And as a matter of fact, even among the engineers, the only ones who are fully competent of doing any real work in this regard are those who have been specially trained in the department in question.

LETTER TO THE EDITOR.

Re "An Interesting Point in Retaining Wall Design."

Sir,—Referring to Mr. E. M. Proctor's letter, which appeared in your issue of February 17th, entitled "An Interesting Point in Retaining Wall Design," I beg to submit the following reply to his enquiries:—

The analysis in Case No. 1 is correct, providing the tension in the back of the wall is taken care of by reinforcing rods. This condition could also exist without the presence of the reinforcing rods, provided the tensile value of the concrete is sufficient to take care of the tensile stress and is not destroyed by the development of cracks.

The analysis in Case No. 2 is correct, providing the tensile value of the concrete is zero and reinforcing rods are omitted. This condition will occur when no reinforcing steel is used and cracks have developed in the back of the wall, thus destroying the tensile value of the concrete. The appearance of cracks in the back of a wall, which is almost sure to be the result if Case No. 2 is used, is exceedingly undesirable and also detrimental to the safety of the wall.

The analysis in Case No. 3 is not correct, as the wall cannot be assumed as a beam under simple bending, because we have in this case both simple bending and direct stress in the section of wall under consideration. The formulae $fc = \frac{M}{\frac{1}{2} k j b d^2}$ is applicable only to reinforced concrete beams under simple bending and therefore cannot be applied in the above case.

R. L. HEARN.

Toronto, February 25th, 1916.

CIVIL SERVICE COMMISSION OF CANADA.

The Civil Service Commissioners announce that applications for two technical clerkships for temporary employment in the topographical branch of the Department of the Interior will be considered from graduates in Applied Science or honor mathematics of some recognized university or those who have passed the final examination for Dominion Land Surveyor or an equivalent examination. Salary will be at the rate of $100 a month and application forms must be filed in the office of the Civil Service Commission, Ottawa, by the 20th of March. Application forms may be obtained by addressing the Secretary of the Commission at Ottawa.

PERSONAL.

D. H. McDOUGALL, Mem.Can.Soc.C.E., has been appointed general manager of the Dominion Steel Corporation.

Capt. F. D. BURPEE, superintendent of the Ottawa Electric Railway, will be major of the 207th Battalion recently authorized to be recruited in Ottawa.

Lieut. R. M. CALVIN, 5th Field Company, Canadian Engineers, has been slightly wounded in a recent engagement in France. Lieut. Calvin is a recent Science graduate from Queen's.

F. W. THOROLD, of Toronto, W. S. LEA, of Montreal, and F. A. DALLYN, engineer of the Ontario Board of Health, have been appointed to report on the Sarnia waterworks.

JOHN COLLINS has been appointed general manager of the Canadian Steam Boiler Equipment Company, Toronto. Mr. Collins was formerly supervising engineer with Gillespie Bros., Toronto.

BERTRAM W. SETON has temporarily severed his connection with the Dominion Engineering and Inspection Company in order to take charge of the adjustment department of the Imperial Munitions Board.

FORREST & LIGHTFOOT have opened an office in Quebec City as engineers and contractors. Both members of the firm have been connected with the National Transcontinental in the construction of shops at Transcona and Quebec.

ALEXANDER C. HUMPHREYS, president of the Stevens Institute of Technology at Hoboken, N.J., and one of the leading engineers of the United States, was the speaker at a Canadian Club luncheon held in the Chateau Laurier, Ottawa, recently.

E. P. ROBERTS, consulting engineer, of Cleveland, Ohio, called at *The Canadian Engineer* office last week on his way to the Cobalt District to report on a water power project which United States capitalists may finance. Mr. Roberts is a past president of the Cleveland Engineering Society and has in the past reported on several other Canadian water power schemes.

FRANK G. WALLACE, of Pittsburgh, Pa., for many years a director of the Canadian Locomotive Company, has accepted the position of managing director of the company, and WILLIAM CASEY, who has held the position of assistant general manager, has been promoted to be manager. The resignation of A. W. WHEATLEY as general manager was announced recently.

CHAS. J. MURPHY, B.A.Sc., A.M.Can.Soc.C.E., who for the past five or six years has been chief engineer of the Crow's Nest Pass Coal Co., The Morrissey, Fernie & Michel Railway, and the Crow's Nest Pass Electric Light & Power Co., has decided to enter private practice, and has opened an office as a consulting engineer, in the Nova Scotia Bank Building, St. Catharines, Ont. Mr. Murphy is an S.P.S. graduate. Previous to going west, he was on the metallurgical staff of the Canadian Copper Company.

The Panama Canal dredging fleet has established a new record for 24 hours, having taken 57,300 cubic yards of earth out of the Gaillard Cut in that time.

The previous high mark was less than 45,000 cubic yards. Of the amount taken out in the record achievement the dredge "Cascadas" alone removed 23,500 cubic yards. The prior record for a dredge was held by the "Paraiso," with 18,000 cubic yards.

OBITUARY.

WILLIAM A. LAVIN, a well-known contractor of Moose Jaw, Sask., was killed a few days ago in an automobile accident at Long Beach, Cal.

WALTER R. LEAVENS passed away at his home in Hallowell Township, Ont., recently, at the age of 59. He had been road surveyor for the township of Hallowell for many years, and had superintended the construction of most of the bridges in that district.

G. G. SHELDON WILLIAMS, of Victoria, B.C., who for 17 years was editor of the British Columbia Mining Exchange and Engineering News, died recently. He was a member of the Canadian Mining Institute and did a great deal of wark in connection with circulating information about British Columbia's mineral wealth.

STEVENSON LAMBE, consulting engineer of New York, died recently. Mr. Lambe, who was in his 79th year, has held many important posts. In 1857 he was appointed city surveyor of New York; later he became chief engineer of sewers. He also planned and built New York's first cable road in 1883. As consulting engineer for the department of works he introduced a system of improved street pavements, which is in extensive use throughout the United States. In 1897 he was appointed consulting engineer of sewerage in connection with the rapid transit tunnel. Mr. Lambe was one of the original members of the American Society of Civil Engineers, having been elected in 1868.

VICTORIA BRANCH, CANADIAN SOCIETY OF CIVIL ENGINEERS.

At the meeting of the branch held on February 22nd a very interesting lecture on "Modern Bridge Architecture" was given by C. E. Fowler, C.E., of Seattle. Mr. Fowler discussed bridges in a non-technical manner, going into the history of bridge building, as well as fully describing modern types of bridges. His address was well illustrated with photographic slides.

CALGARY BRANCH, CANADIAN SOCIETY OF CIVIL ENGINEERS.

The Calgary branch recently gave a dinner in honor of C. D. Howe, chief engineer of the Dominion Government Grain Commission. Following the dinner, Mr. Howe gave an address on the construction and purposes of government internal storage elevators, illustrating it with views of the elevators, particularly of the Calgary one.

At a meeting of the branch held on February 24th, an innovation in the way of a "ladies' night" was tried, and all present agreed that it was a success.

After the dinner, Dr. J. G. Rutherford, superintendent of agriculture and animal industry, C.P.R., Department of Natural Resources, addressed the meeting. Dr. Rutherford chose "Some Thoughts on the Present World Situation" as his subject. He stated that the world, as we know it, is barely one hundred years old by reason of the remarkable development that has taken place in the last hundred years.

L. K. Comstock and Company, contracting engineers, have opened their Canadian offices in Room No. 609 New Birks Building, Montreal, with Douglas-Milligan Company in charge as their representatives.