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REINFORCED CONCRETE DESIGN

DESIGNING REINFORCED CONCRETE COLUMNS, FLOOR SLABS, BEAMS, GIRDERS AND ARCH SECTIONS WITH THE AID OF NEW GRAPHIC DIAGRAMS.

By **DAVID A. MOLITOR, C.E., Mem.Am.Soc.C.E.**

WHILE diagrams do not usually afford the same accuracy obtainable from analytic computations, yet no extreme accuracy is ever required in concrete design since the nature of the material and its changes with age render concrete far more uncertain than steel. The best standard steel specifications permit a 10% variation in ultimate strength and about 2% variation in the cross-sections of finished members, to say nothing of the inaccuracies in the assumed dead and live loads. With concrete, these variations are admittedly greater, and hence no extreme accuracy can be exacted in reinforced concrete designs.

Therefore, if graphic diagrams can be made to yield results within 5% of computed values, no valid objection can be raised against their use. Generally the results will be much better and gross mistakes are practically eliminated, because each result can be so easily checked by a repetition of the readings taken.

The formulae here used are based on the usual assumption that the concrete takes no tension, and that the compressive stress in the concrete varies directly as the distance from the neutral axis.

Where double reinforcement is employed, it will always be taken symmetric, and the compression will be carried by both steel and concrete, while tension, if present, will be carried by the steel alone.

These assumptions accord well with the facts, and lead to the formulae given below, which are now almost universally accepted.

The allowable working stresses for concrete, as given in Table II., are based on Class A concrete, consisting of granite, trap rock, gravel or hard limestone in sizes not exceeding ¾-inch for small form work to 1½-inch for heavy beams and columns. This stone aggregate is mixed with clean, sharp sand and Portland cement in the stated proportions.

As a specification for quality of Class A concrete, 12-inch cubes, when tested to crushing, must develop an ultimate compressive strength not less than the values given in Table I. for one month and six months after mixing.

The unit working stresses adopted are based on a factor of safety of five, or slightly over, for concrete at the age of 6 months, which is considered good conservative practice. The modular ratio n is taken as 15 for average working conditions.

TABLE I.

ULTIMATE COMPRESSIVE STRENGTH (Pounds per Square Inch), Class A Concrete.

Concrete mixture, parts by volume.	1 month.	6 months.
1:1.5:3	2,600	3,400
1:2 :4	2,200	3,200
1:2.5:5	2,000	3,000
1:3 :6	1,800	2,800

The column formulae used are those given in Turneaure and Maurer's "Principles of Reinforced Concrete Construction, 1909," page 271. Diagram 1 gives the allowable unit working stress for columns with longitudinal reinforcement only, and for columns with hooped or spiral circumferential reinforcement, with proper reduction for columns longer than 10 d. The safe working stress for the concrete alone is given in Table II. under heading columns. See examples for illustration.

The beam and slab formulae employed are given on the annexed sheets and are also to be found in the above-named treatise. These are based on the fundamental equation $k = \sqrt{2pn + (pn)^2} - pn$, first proposed by Coignet and Tedesco, for locating the position of the neutral axis in terms of the modular ratio n and the steel ratio p, where k is the fractional part of the net depth d which the neutral axis is below the extreme fibre on the compression side. Diagrams 2, 3, 4 and 5 furnish beam and steel dimensions for all externally applied bending moments up to one million foot-lbs. for slabs or beams one foot wide. These dimensions are based on such values of p for which the moment of resistance of the concrete equals the moment of resistance of the steel and hence lead to economic designs. These diagrams are, of course, applicable to beams of any width merely by correcting the applied bending moment to the equivalent for 1 foot width.

Direct stress and bending. Diagrams 6, 7 and 8.—The formulae for solving this problem are quite complex, hence the solution by curves should prove most welcome to designers. The lettered dimensions given on Diagrams 6 and 7 are self-explanatory. The general problem is divided into two cases: Case 1, symmetric reinforcement with no tension, Diagram 6; Case 2, symmetric reinforcement with tension, Diagrams 7 and 8. In each case the thickness of insulation is taken as $d_1 = D/10$, making $a = 0.4\ D$. This will usually give good values, even for very large values of D. The reason for adopting this fixed relation is to simplify the formulae which could not otherwise be solved by curves owing to the extra variable d_1.

The curves cover any case of eccentrically applied loading, as for columns, piers or arch sections where N = the resultant thrust normal to the section and v = the eccentricity of this thrust.

Diagram 6 gives everything required for designing a section or for finding the stresses in a given design.

Diagram 7 gives all information for designing a section, while the additional diagram 8 is added to find the stresses in a given design.

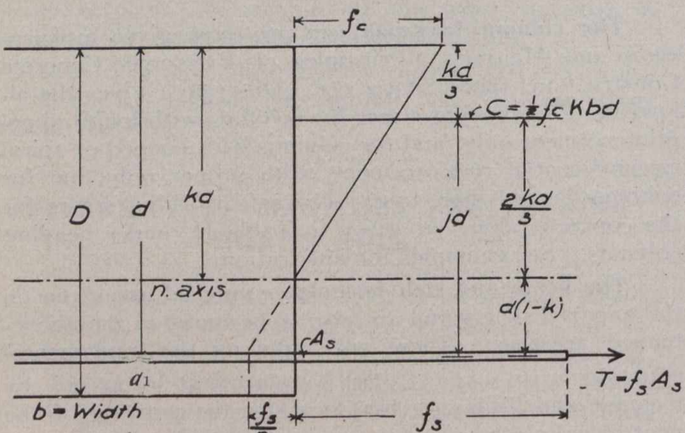
ALLOWABLE UNIT STRESSES AND BEAM FORMULAE.

TABLE II.
PHYSICAL CONSTANTS, CLASS A CONCRETE

Mixture Concrete.	E lbs. per sq. in. (6 months).	ϵ for 1° F.	Ult. comp. lbs. per sq. in. 12" Cubes (6 months)	Working stresses, lbs. per sq. in. Compression.	Columns.	Tension.	Shear.	Bond.	$n = E_s/E_c$
1:1.5:3	3,500,000	0.000,0050	3,400	650	550	0	45	Smooth rod, 60 lb., develop-	15
1:2:4	3,000,000	0.000,0055	3,200	600	500	0	40	ed in 65 diam. Deformed	for average
1:2.5:5	2,500,000	0.000,0060	3,000	550	450	0	35	bars, 150 lb.,	working
1:3:6	2,000,000	0.000,0065	2,800	500	400	0	30	developed in 25 diam.	
Steel	30,000,000	0.000,0067	65,000	15,000	10,000	15,000	10,000	for O bars.	conditions.

Weight of plain concrete = 144 lbs. per cu. ft. 1% reinforcement = 132 lbs. per cu. yd. = 4.9 lbs. per cu. ft.

Rectangular Beams in Simple Flexure, Neglecting Tension in the Concrete.



Moment of resistance of the concrete

$$= M_c = \tfrac{1}{2} f_c k j b d^2 = Cjd = R_c b d^2.$$

Moment of resistance of the steel

$$= M_s = f_s p j b d^2 = Tjd = R_s b d^2.$$

The smaller value of M or R governs the strength of the beam.

Steel ratio:

$$p = \frac{A_s}{bd};\ k = \sqrt{2pn + (pn)^2} - pn;\ \text{and } j = 1 - \frac{k}{3}.$$

$$\text{Also, } A_s = pbd;\ f_c = \frac{2M}{kjbd^2} = \frac{2R_c}{kj};\ \text{and } f_s = \frac{M}{pjbd^2} = \frac{M}{jd\,A_s} = \frac{R_s}{pj},$$

where M = moment of the external forces.

$$\text{When } M_c = M_s, \text{ then } R_c = R_s \text{ and } p = \frac{1}{\frac{2f_s}{f_c}\left(\frac{f_s}{nf_c} + 1\right)}$$

Insulation $d_1 = \frac{1}{2}\sqrt{D}$ about, and $D = d + d_1$. Min $d_1 = \frac{3}{4}$".

TABLE III.
WORKING VALUES FOR CLASS A CONCRETE.

Mixture Concrete.	$\frac{f_s}{f_c}$	p for $n=15$, $M_s = M_c$	Steel per cu. ft.	j	k	$R_c = \frac{1}{2} f_c k j$	$R_s = f_s p j$	Moments of external loading. Nature of supports.	Uniform load.	Concentr load.
1:1.5:3	23.1	0.0085	4.17 lbs.	0.869	0.393	110.9	110.9			
1:2:4	25	0.0075	3.68 lbs.	0.875	0.375	98.4	98.4	Simple beam, 2 supports	$M = pl^2/8$	$M = Pl/4$
1:2.5:5	27.3	0.0065	3.19 lbs.	0.882	0.355	86.0	86.0	One end continuous	$M = pl^2/10$	$M = Pl/5$
1:3:6	30	0.0056	2.74 lbs.	0.888	0.335	74.5	74.5	Continuous beam	$M = pl^2/12$	$M = Pl/6$

The cross-section of a beam, for a given bending moment M, is found from $bd^2 = \frac{M}{R}$ and when R_c and R_s are not equal then the smaller value governs. The graphic diagrams furnish values of d, A_s and bar sizes for all values of M and R, when $n = 15$. The points where horizontal reinforcement may be turned up can be found as for plate girder flanges.

For floor slabs, the maximum bar spacing should never exceed d.

Shear reinforcement. Let V = total shear on any vertical section of width b and net depth d. Also let a = the required stirrup area for this section, when stirrups are spaced s lengthwise of the beam.

The average vertical shear per sq. in. of section is $v = V/bd$, of which $0.7\,v$ is assumed to be carried by the stirrups and $0.3\,v$ by the concrete. The stress in stirrup rods at one section is $Q = 0.7\,vbs$ for vertical stirrups, giving the required stirrup area for a single section as

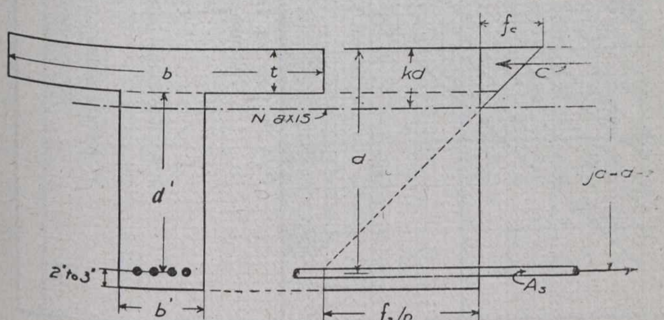
$$\left.\begin{aligned} a &= \frac{Q}{f_s} = \frac{0.7\,vbs}{10{,}000} = \frac{0.7\,Vs}{10{,}000\,d} \text{ for vertical stirrups} \\ a &= \frac{0.7\,Q}{f_s} = \frac{0.5\,vbs}{10{,}000} = \frac{0.5\,Vs}{10{,}000\,d} \text{ for stirrups at } 45^\circ \end{aligned}\right\}$$

assuming that the stirrups carry 0.7 of the shear and that none of the horizontal reinforcement is bent up to carry shear.

The horizontal stirrup spacing s should not exceed $0.6\,d$.

It is preferable to turn up a portion of the horizontal reinforcement near the ends to carry the shear, rather than to employ stirrups for this purpose.

T-Beams in Simple Flexure.



l = span of beam; l' = span of floor slab.

Limits in design:

$b = \frac{f_s A_s}{f_c t\,(1 - t/2kd)}$ or $\not> 8t$ and $t > l'/30$.

$d = l/12$ to $l/10$ and $d' = \sqrt{\frac{60\,M}{f_s b'}} - \frac{t}{2}$ for min. cost.

b' must be chosen to suit the rod spacing, allowing 2.5 diam. for round and 3 diam. for square rods.

The min. web area $= b'd = \frac{V_{max}}{100}$, allowing 100 lbs. per sq. in. shear on the gross area.

Find $k = \sqrt{2\,pn + (pn)^2} - pn$, noting that $p = \frac{A_s}{bd}$, not $\frac{A_s}{b'd}$. When $k < \frac{t}{d}$ we have Case I

Case I.—When the neutral axis falls inside the slab or flange, making k < t/d.

The formulæ for rectangular beams apply here and the steel usually governs, while f_c will be small. Hence approximately $M_s = f_s A_s\,(d - \frac{t}{3})$ and $M_c = \frac{1}{2} f_c kjbd^2$, giving $A_s = \frac{M}{f_s\,(d - t/3)}$ or $f_s = \frac{M}{A_s\,(d - t/3)}$ and $f_c = \frac{2\,M}{kjbd^2} = \frac{2\,R_c}{kj}$ where M = moment of external forces, k as above and $j = 1 - k/3$.

Case II.—When the neutral axis falls in the web, neglecting compression in the web.

$$k = \frac{nA_s + \frac{bt^2}{2d}}{nA_s + bt} = \frac{pn + \frac{1}{2}(\frac{t}{d})^2}{pn + t/d} \text{ and } z = \frac{(3\,k - 2\frac{t}{d})\frac{t}{3}}{2k - t/d} \text{ also } j = 1 - \frac{z}{d} = \frac{1 - \frac{t}{d} + \frac{1}{3}(\frac{t}{d})^2 + \frac{(t/d)^3}{12\,pn}}{1 - t/2d}$$

$M_s = f_s A_s(d - z)$ and $M_c = f_c\,(1 - \frac{t}{2kd})\,(d - z)\,bt$, the smaller value governing.

$$\text{Finally } A_s = \frac{M}{f_s(d - z)};\ f_s = \frac{T}{A_s} = \frac{M}{A_s(d - z)};\text{ and } f_c = \frac{f_s k}{n(1 - k)} = \frac{pf_s}{(1 - \frac{t}{2kd})\frac{t}{d}} = \frac{M}{(1 - \frac{t}{2kd})\,(d - z)bt}.$$

For designing curves $R = \frac{M}{bd^2} = f_c\,(1 - \frac{t}{2kd})\,\frac{t}{d}.j.$

Approximately, $M_s = f_s A_s(d - \frac{t}{2})$; $M_c = \frac{1}{2} f_c bt\,(d - \frac{t}{2})$; $C = T = \frac{M}{d - t/2}$.

$f_c = \frac{2C}{bt}$ and $f_s = \frac{T}{A_s}$. Assume $jd = d - z = \frac{7}{8}d$.

Shear Reinforcement.—The required stirrup area at a single section becomes $a = \frac{0.7\,Vs}{10{,}000\,(d - t/2)}$ for vertical stirrups and $a = \frac{0.5\,Vs}{10{,}000\,(d - t/2)}$ for stirrups at 45°.

Slab Reinforcement in Two Directions.—

$r = \frac{l^4}{l^4 + l_1^4}$ = proportion of load carried by the short span l_1 for rectangular slabs.

for l/l_1 =	1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0
r =	0.5	0.6	0.67	0.74	0.79	0.83	0.87	0.89	0.91	0.93	0.94

REINFORCED CONCRETE COLUMNS.

Let P = safe axial load in pounds when $l/d \leqq 10$.

P' = safe axial load in pounds for $l/d = 10$ to 30.

A = effective area of column in sq. inches.

A_s = sectional area of longitudinal steel in sq. inches.

A'_s = sectional area of circumferential steel in sq. inches.

$p = \frac{A_s}{A}$ and $p' = \frac{A'_s}{A}$ = steel ratios.

$n = \frac{E_s}{E_c} = 15$ for average working conditions.

f_c = safe unit working stress for concrete in columns.

l = unsupported length of column in inches.

d = least dimension of effective area in inches.

d_1 = insulation = 2″ all around for fire protection.

$s = \frac{ac}{p'A}$ = pitch of circumferential steel, where a = area of one hoop and c = length of a single hoop = $4d$ for square and πd for round columns.

The safe axial load on columns shall be determined by the following formulae:—

1. Piers without reinforcement:

$P = Af_c$ for $l/d \leqq 6$.

2. Columns with longitudinal reinforcement only:

$P = Af_c[1 + (n - 1)p]$ for $l/d \leqq 10$ and $p = 0.02$ to 0.04.

Longitudinal bars must be tied at intervals not exceeding 1 ft.

3. Columns with longitudinal and circumferential reinforcement:

$P = Af_c[1 + 14p + 30p']$ for $l/d \leqq 10$ and $p = 0.02$ to 0.04, $p' = 0.01$.

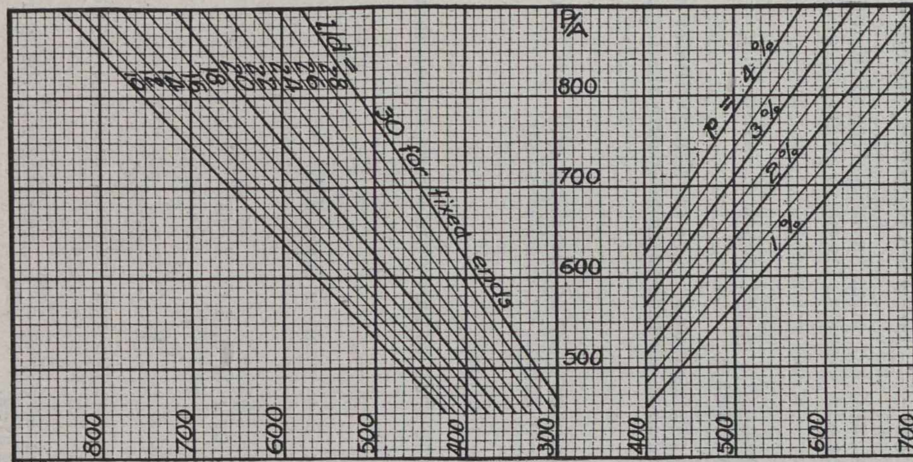
Spacing of circumferential bars, $s = d/8$ to $d/5$.

4. Columns longer than 10 d and shorter than 30 d:

For fixed ends. For hinged ends.

$$P' = \frac{P}{1 + \frac{1}{1600}\left(\frac{l}{d}\right)^2}, \quad P' = \frac{P}{1 + \frac{1}{800}\left(\frac{l}{d}\right)^2}.$$

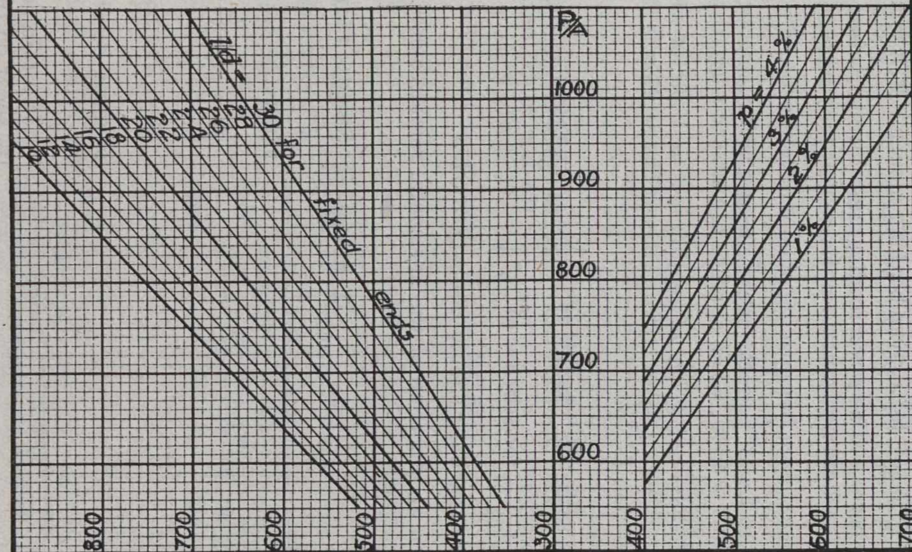
Diagram 1.—REINFORCED CONCRETE COLUMNS.



P'/A lbs. per sq. in. f_c lbs. per sq. in.

Columns with Longitudinal Reinforcement only.

$P/A = f_c[1 + 14p]$ for $l/d \leqq 10$



P'/A lbs. per sq. in. f_c lbs. per sq. in.

Columns with 1% Circumferential Reinforcement.

$P/A = f_c[1.3 + 14p]$ for $l/d \leqq 10$

Circumferential bars spaced $\frac{1}{8}$ to $\frac{1}{5}$d.

P/A = allowable unit stress in pounds per sq. inch of effective area, for columns shorter than 10d.

P'/A = allowable unit stress in pounds per sq. inch of effective area, for columns longer than 10d and shorter than 30d.

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Diagram 2.—SLABS IN SIMPLE FLEXURE.

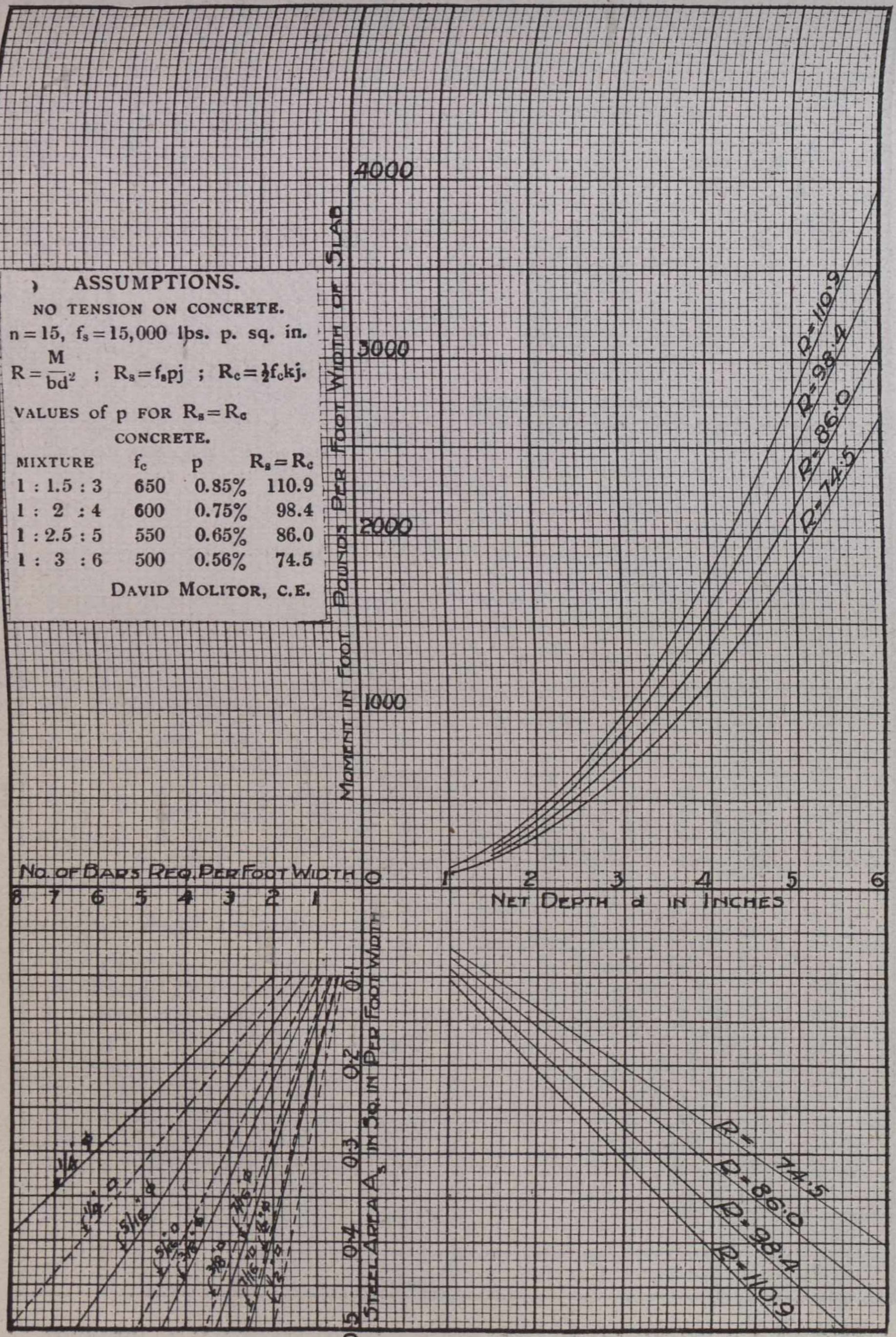


Diagram 3.—RECTANGULAR BEAMS IN SIMPLE FLEXURE.

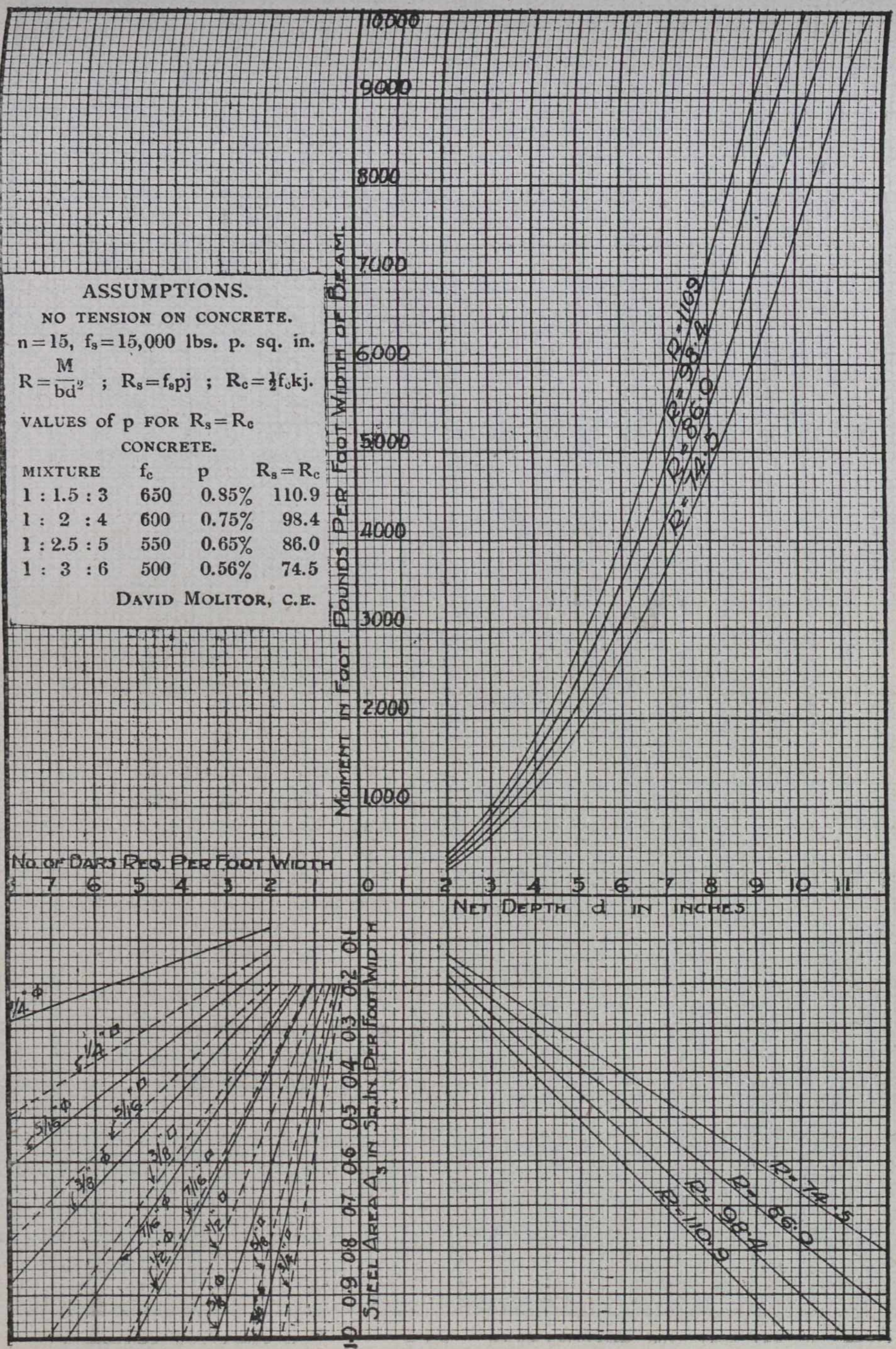


Diagram 4.—RECTANGULAR BEAMS IN SIMPLE FLEXURE.

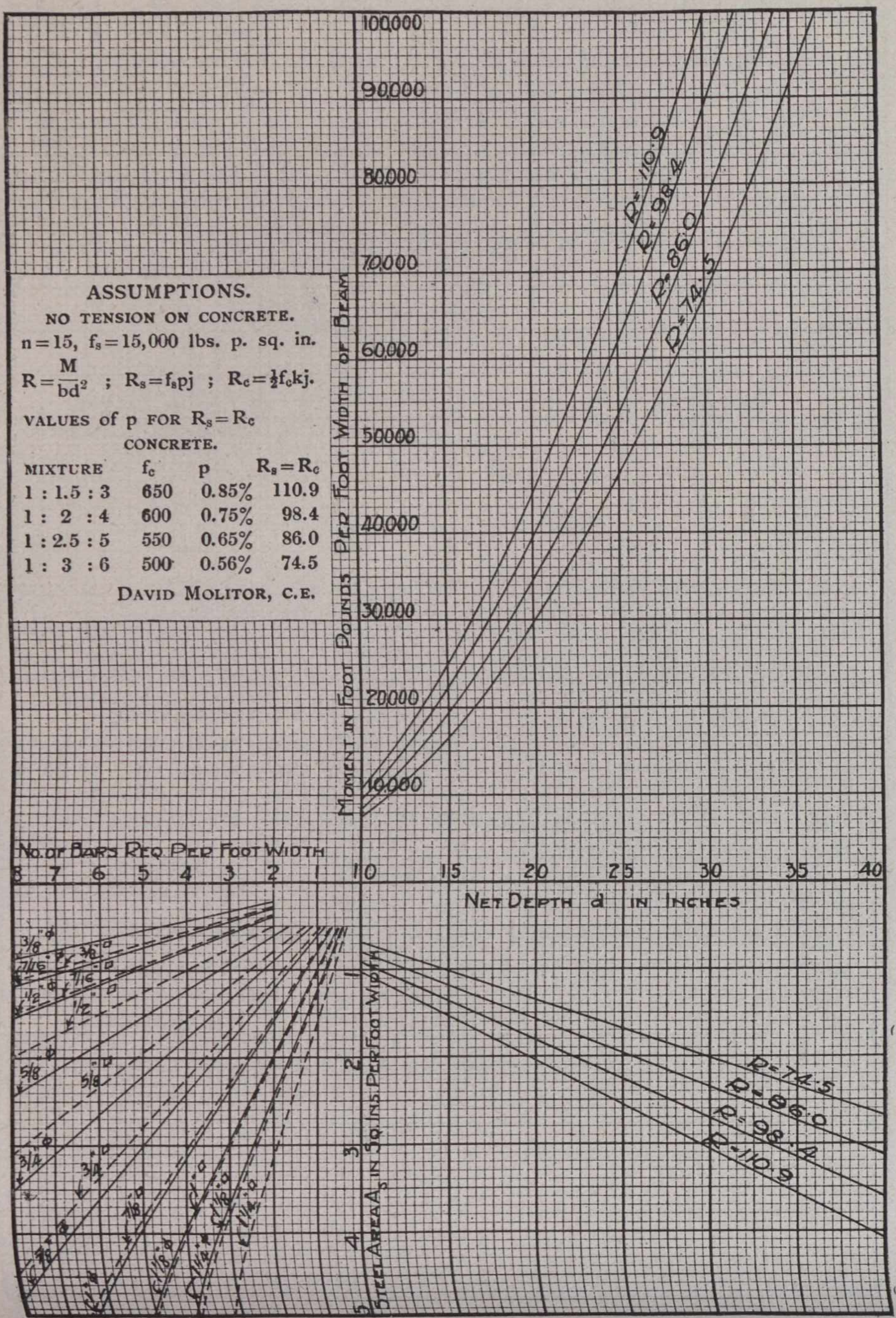
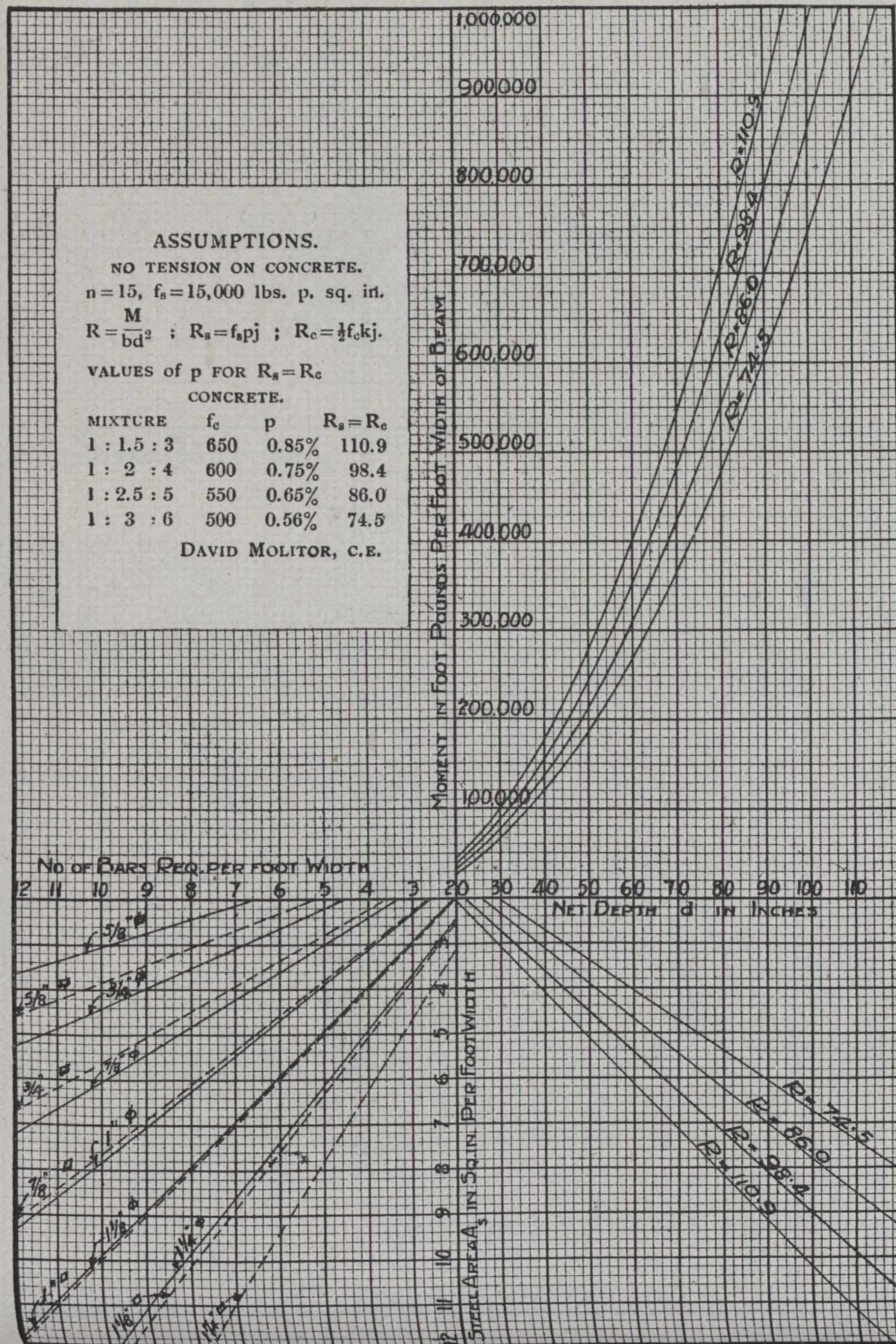
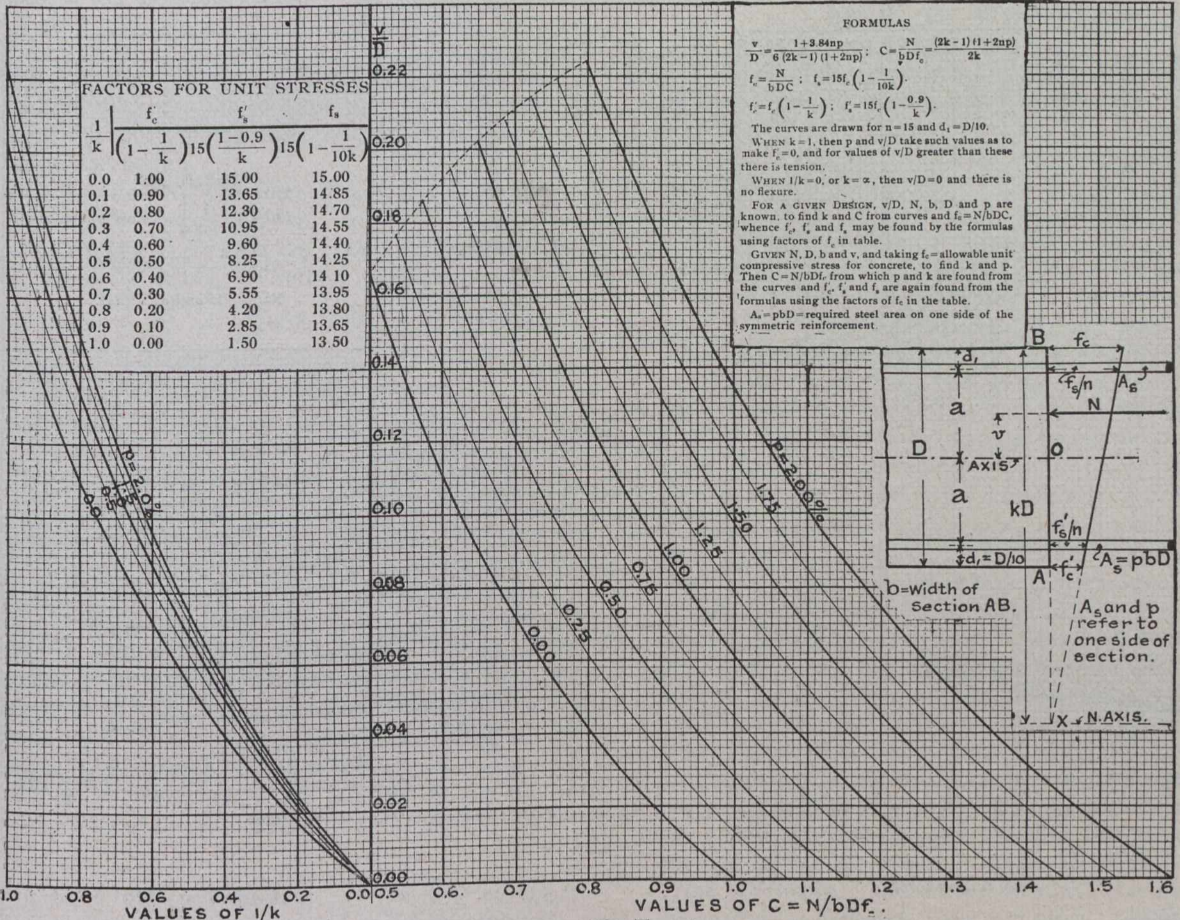


Diagram 5.—RECTANGULAR BEAMS IN SIMPLE FLEXURE.



DIRECT STRESS AND FLEXURE SYMMETRIC REINFORCEMENT.



FACTORS FOR UNIT STRESSES

$\frac{1}{k}$	f_c' $\left(1-\frac{1}{k}\right)$	f_s' $15\left(\frac{1-0.9}{k}\right)$	f_s $15\left(1-\frac{1}{10k}\right)$
0.0	1.00	15.00	15.00
0.1	0.90	13.65	14.85
0.2	0.80	12.30	14.70
0.3	0.70	10.95	14.55
0.4	0.60	9.60	14.40
0.5	0.50	8.25	14.25
0.6	0.40	6.90	14 10
0.7	0.30	5.55	13.95
0.8	0.20	4.20	13.80
0.9	0.10	2.85	13.65
1.0	0.00	1.50	13.50

FORMULAS

$\frac{v}{D}=\frac{1+3.84np}{6(2k-1)(1+2np)}$; $C=\frac{N}{bDf_c}=\frac{(2k-1)(1+2np)}{2k}$

$f_c=\frac{N}{bDC}$; $f_s=15f_c\left(1-\frac{1}{10k}\right)$.

$f_c'=f_c\left(1-\frac{1}{k}\right)$; $f_s'=15f_c\left(1-\frac{0.9}{k}\right)$.

The curves are drawn for $n=15$ and $d_1=D/10$.

WHEN $k=1$, then p and v/D take such values as to make $f_c'=0$, and for values of v/D greater than these there is tension.

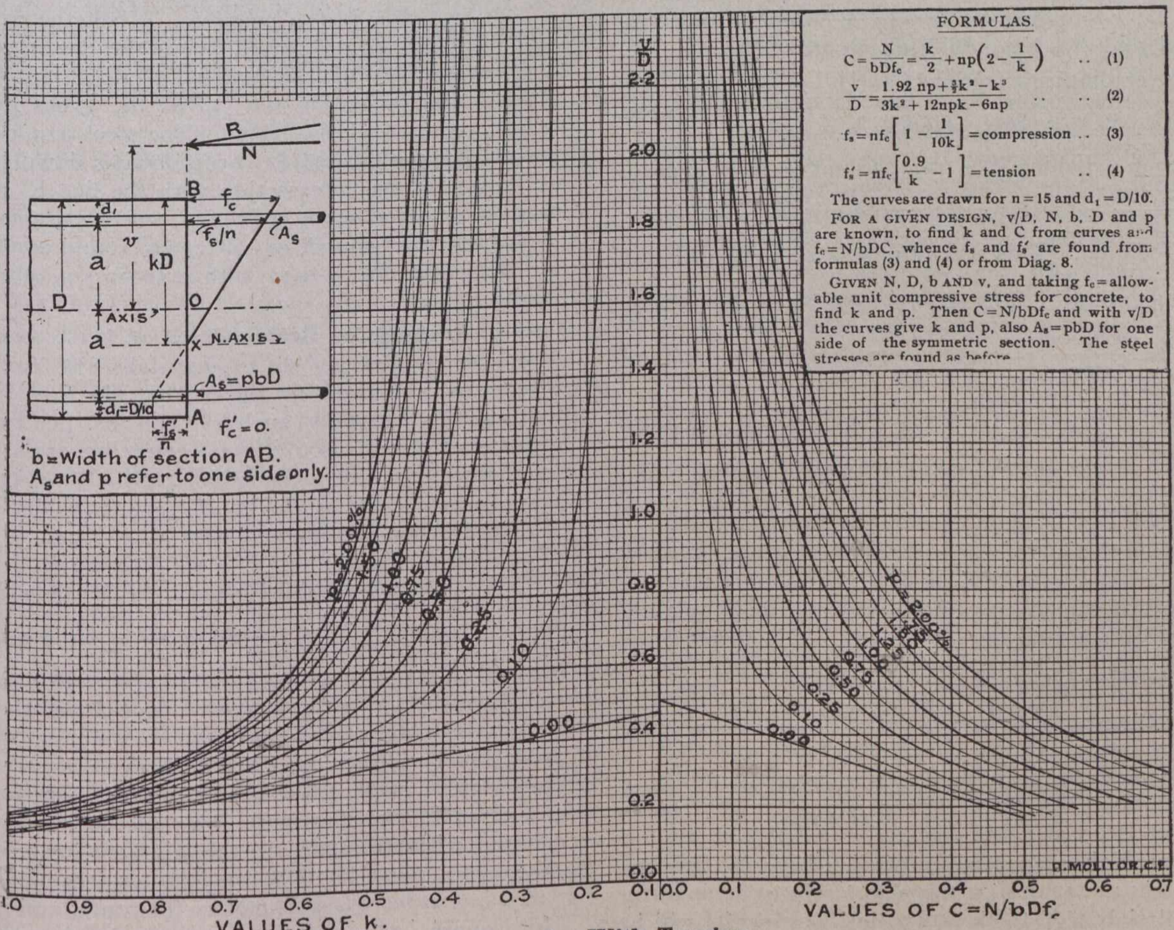
WHEN $1/k=0$, or $k=\infty$, then $v/D=0$ and there is no flexure.

FOR A GIVEN DESIGN, v/D, N, b, D and p are known, to find k and C from curves and $f_c=N/bDC$, whence f_c', f_s' and f_s may be found by the formulas using factors of f_c in table.

GIVEN N, D, b and v, and taking f_c = allowable unit compressive stress for concrete, to find k and p. Then $C=N/bDf_c$ from which p and k are found from the curves and f_c', f_s' and f_s are again found from the formulas using the factors of f_c in the table.

$A_s=pbD$ = required steel area on one side of the symmetric reinforcement.

Diagram 6.—No Tension.



FORMULAS

$C=\frac{N}{bDf_c}=\frac{k}{2}+np\left(2-\frac{1}{k}\right)$ .. (1)

$\frac{v}{D}=\frac{1.92\,np+\frac{3}{2}k^2-k^3}{3k^2+12npk-6np}$ .. .. (2)

$f_s=nf_c\left[1-\frac{1}{10k}\right]$ = compression .. (3)

$f_s'=nf_c\left[\frac{0.9}{k}-1\right]$ = tension .. (4)

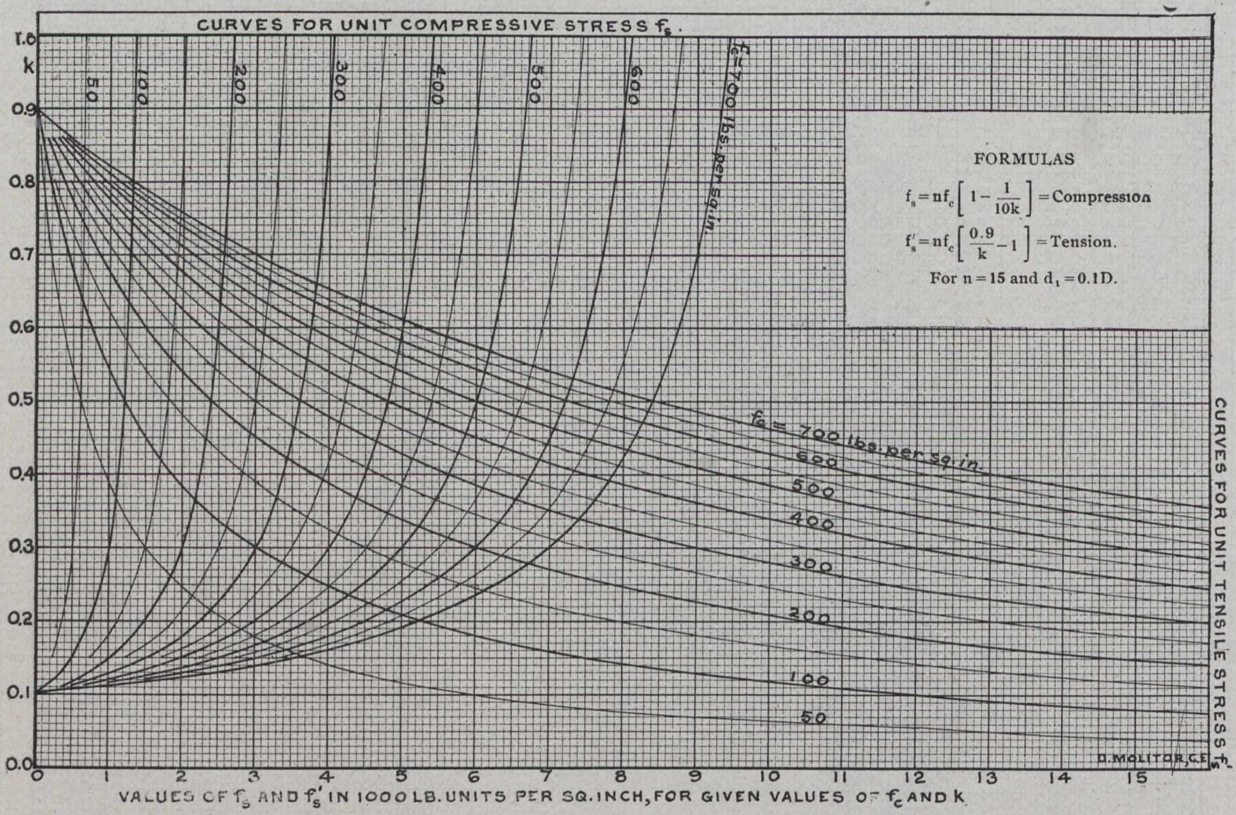
The curves are drawn for $n=15$ and $d_1=D/10$.

FOR A GIVEN DESIGN, v/D, N, b, D and p are known, to find k and C from curves and $f_c=N/bDC$, whence f_s and f_s' are found from formulas (3) and (4) or from Diag. 8.

GIVEN N, D, b AND v, and taking f_c = allowable unit compressive stress for concrete, to find k and p. Then $C=N/bDf_c$ and with v/D the curves give k and p, also $A_s=pbD$ for one side of the symmetric section. The steel stresses are found as before.

Diagram 7.—With Tension.

Diagram 8: Case 2.—DIRECT STRESS AND FLEXURE, SYMMETRIC REINFORCEMENT WITH TENSION.



EXAMPLES.

1. Column.—Let $l = 12$ ft., fixed ends, $P = 100,000$ lbs. and $f_c = 500$ lbs. per sq. in. for 1 : 2 : 4 concrete as per Table II. Required to design a column using 3% longitudinal and 1% hoop reinforcement.

The lower diagram (1) gives for $f_c = 500$, $p = 3\%$ and $p' = 1\%$, $P/A = 860$. Hence $A = \frac{P}{860} = \frac{100,000}{860} = 116.3$ sq. in., making $d = 12.2''$ round or $10.8''$ square, and $\frac{l}{d} = \frac{12 \times 12}{12.2} = 11.8$ for the round or $\frac{144}{10.8} = 13.3$ for the square column. This will require a reduction in P/A to P'/A.

Thus, for $P/A = 860$ and $l/d = 11.8$, $P'/A = 800$, making $A = \frac{100,000}{800} = 125$ sq. in., or $12.6''$ round, to which add $4''$ for insulation.

For $P/A = 860$ and $l/d = 13.3$, $P'/A = 775$, making $A = \frac{100,000}{775} = 129$ sq. in., or $11.4''$ square, to which add $4''$ for insulation.

Round column: $A_s = pA = 0.03 \times 125 = 3.75$ sq. in.; use four $1\frac{1}{8}$ round bars.

Square column: $A_s = pA = 0.03 \times 129 = 387$ sq. in.; use four $1\frac{1}{8}$ round bars.

Hoop spacing using $\frac{1}{4}$-in. round steel,

$$\text{for round column } s = \frac{\pi a d}{p' A} = \frac{3.14 \times 0.049 \times 12.6}{0.01 \times 125} = 1.55 \text{ in.}$$

$$\text{for square column } s = \frac{4 a d}{p' A} = \frac{4 \times 0.049 \times 11.4}{0.01 \times 129} = 1.74 \text{ in.}$$

2. Floor Slab.—Span 6 ft., live load 200 lbs. per sq. ft., concrete 1 : 2 : 4, design the slab. The dead load for an assumed slab $4\frac{1}{4}''$ thick at 150 lbs. per cu. ft. will be 53 lbs. per sq. ft., making the total load 253 lbs. The bending moment for non-continuous span is $253 \times 6 \times 6/8 = 1,138$ ft.-lbs. From Diagram 2 find $R = 98.4$ and $p = 0.75\%$ for 1 : 2 : 4 concrete. Then the net depth of slab is found as $d = 3.4''$, being the depth given below the intersection of the line $M = 1,138$ and $R = 98.4$. Adding the insulation $0.9''$ gives the gross depth $D = 3.4 + 0.9 = 4.3''$, say, $4\frac{3}{8}''$. The steel required is found on the same diagram (2) by following down on the line $d = 3.4''$ to the intersection with the line $R = 98.4$ and find $A_s = 0.308$ sq. in. Finally, select suitable bar sizes, as 4 bars $5/16''$ round, or 3 bars $\frac{3}{8}''$ round per foot width, or steelcrete No. 3-9-30 with 0.30 sq. in. area per foot width.

3. Rectangular Beam.—Span 16 ft c.c. end bearings, live load 200 lbs. per sq. ft., for slabs 6 ft. c.c. of beams. Concrete 1 : 2 : 4, design the beam. The live load per foot of beam is then $6 \times 200 = 1,200$ lbs. per lin. ft. The dead load of slab is $6 \times 53 = 318$ lbs., and the weight of the beam will be assumed as 420 lbs., making the total load to be carried by the beam, say, 1,940 lbs. per lin. ft. Hence for non-continuous beam $M = 1,940 \times 16 \times 16/8 = 62,080$ ft. lbs.

Assume the width of beam $b = 18''$ then the moment per foot width will be $62,080 \times \frac{12}{18} = 41,400$ ft.-lbs., for which Diagram 4 gives $d = 20.4''$ for $R = 98.4$. The total depth $D = 20.4 + 2.25 = 22.65''$ and the steel area from the diagram becomes $A_s = 1.85$ sq. in. per foot width of beam, giving $1.85 \times 18/12 = 2.77$ sq. in. for a beam 18 in. wide. Use five $\frac{3}{4}''$ square bars for the beam as given by the diagram. The number of bars is really chosen without regard to the width of beam except that the beam must be wide enough to accommodate them, allowing at least 2.5 diameters for round and 3 diameters for square bars, centre to centre. In the present beam the rods would be spaced 3½" c. to c., and three of them should be bent up, bending up two at about ⅙ of the span from each end, and one at about ⅓ of the span from the ends. The maximum end shear = 8 × 1,940 = 15,520 lbs., which is almost entirely carried by the concrete allowing 40 lbs. per sq. in. No stirrups are required, and, in fact, none of the longitudinal steel would receive any shear in this particular beam.

4. Arch Section, No Tension.—From the arch analysis we have given N = 553,400 lbs., v = 10", D = 60", b = 24" and make f_c = 600 lbs. per sq. in. for 1 : 2 : 4 concrete. Find A_s, k, f'_c, f'_s and f_s.

$$\text{Now, } v/D = \frac{10}{60} = 0.1668, \text{ also } C = N/bdf_c = \frac{553{,}400}{24 \times 60 \times 600} = 0.641.$$

From Diagram 6 find p = 0.66% for v/D and C, and also 1/k = 0.86 or k = 1.163 for v/D = 0.1668 and p = 0.66%. Then, from the table on Diagram 6 find the factors of f_c for 1/k = 0.86. These are interpolated to get the following stresses: $f'_c = 0.14f_c$ = 84 lbs. per sq. in., $f'_s = 3.39f_c$ = 2,030 lbs. per sq. in., and f_s = 13.7 f_c = 8,230 lbs. per sq. in. The required steel area is A_s = pbD = 0.0066 × 24 × 60 = 9.5 sq. in. on each side of the section. This problem shows that it is very uneconomical to use compressive steel to bring down the compression in the concrete to a safe allowable stress. The section should have been widened to made b = 36".

5. Arch Section with Tension.—Given N = 553,400 lbs., v = 20", D = 60", b = 36" and f_c = 600 lbs. per sq. in. for 1 : 2 : 4 concrete. Find A_s, k, f'_s and f_s.

Now,

$$v/D = \frac{20}{60} = 0.333 \text{ and } c = N/bDf_c = \frac{553{,}400}{36 \times 60 \times 600} = 0.427.$$

Diagram 7 then gives p = 0.71% and k = 0.72, whence A_s = pbD = 0.0071 × 36 × 60 = 15.33 sq. in. The steel stresses for k and f_c are found from Diagram 8 as f'_s = 2,300 lbs. per sq. in. tension and f_s = 7,760 lbs. per sq. in. compression.

ROAD EXPENDITURE IN SIMCOE COUNTY, ONT.

In his report for 1914 County Engineer Campbell showed an expenditure of $52,337 on the road system last year. Of this amount over $31,000 was spent on permanent roads, including bridges, culverts, materials, etc. Unscreened gravel and crushed stone from local pits ¼ to 4½ miles distant was used for metalizing. Bridges were completed as follows:—Nicolston on Essa-Tecumseth town-line, $8,434.68; Draper, on Gwillimbury-Tecumseth line, $3,804.94; Comartin, in Sunnidale, $2,415.78; Graham, in Oro, $2,063.35; Coldwater, $4,499.38; McNall, in Nottawasaga, $2,719.61; and Tiny, $1,453.95.

Rutile is used in electrodes for arc lamps, for making ferro-titanium for use in cast iron and steel, in ceramics and chemicals. Concentrated rutile sells for $50 to $400 a ton, depending on quality, quantity, and preparation.

A new malleable alloy, consisting chiefly of aluminium, has been invented by Thomas A. Bayliss, of Warwick, and Byron George Clark, of London. The composition varies between the limits of aluminium, 80 to 99; zinc, .00001 to 19.999; and cadmium, .001 to 10 per cent. Two particular alloys given as examples are: 1. Aluminium 91, zinc 8, cadmium 1 per cent. 2. Aluminium 88, zinc 10, cadmium 2 per cent. The alloy is claimed to be extremely tough and strong and highly malleable when cast in chill moulds for rolling.

TREATMENT OF SEWAGE BY AERATION.

By R. O. Wynne-Roberts.

In the Engineering Record of February 20th, 1915, there is a reference made to the British research work in connection with sewage treatment by aeration, and whilst it is "apparent that in sanitary engineering this treatment has been the subject of transcendent interest in Great Britain during the past year," the subject is equally absorbing to engineers in North America, for we are informed by the above journal that the United States Public Health Service and the Baltimore Sewerage Commission are to co-operate in a series of experiments on similar lines.

By the same post the writer received a copy of the 1913 annual report of the Massachusetts State Board of Health, which contains an interesting account of studies made at the Lawrence experiment station upon the aeration of sewage in a new form of tank.

The aeration of sewage is an old process, which has been tried in many places and in many ways, but up to a short time ago, no definite and permanent results commensurate with the cost were obtained. Dr. Angus Smith (London) in 1882 found that aerated sewage was less putrefactive and formation of nitrates could be facilitated. Other experimenters endeavored to carry the process of aeration to a successful and practicable issue. Messrs. Phelps and Black, in their comprehensive study of New York harbor conditions in 1910, made some experiments as to the degree of stability which could be attained, with the result that they suggested that the process should be applied on a large scale. Dupre, Dibden, Brown, Mason, Hine, Waring, Lowcock, Brauze, Helbig, Hartland, Kaye-Parry, Fowler, Ardern, Lockett, Melling, Mumford and Duckworth have each contributed their share to our present knowledge on this subject.

Returning to the Lawrence experiments first: It is unnecessary to relate their history and object, as the full published reports can be obtained and studied. Suffice it to state that a noticeable clarification of sewage by aeration was observed. It was further noticed that when green algal growths occurred in the aerated sewage it became saturated with oxygen. So, in April, 1912, experiments were started in the laboratory and on January 2, 1913, a new aeration tank was built. This tank was similar to Dibden's slate beds. It had horizontal layers of roofing slates with one-inch concrete separation blocks between for the deposition and collection of suspended and colloidal matters. Air was pumped through a perforated pipe on the bottom of the tank in quantities varying from 200,000 cubic feet per hour per million U.S. gallons down to 100,000 cubic feet. In July the slates were rearranged, this time vertically, acting like Travis's colloidon or Martin's cellular upward filters, and the volume of air blown in was reduced as low as 25,000 cubic feet per hour per million U.S. gallons.

Messrs. Clark and Gage, who had charge of the experiments, state that the "various changes in the methods of operating this tank produced far less difference in the efficiency of the process than might be expected from the wide difference in the period of aeration and in the volume of air used at different times." They state, however, that "the greatest efficiency was obtained during the second period, when the sewage was aerated for ten hours with air equivalent to 200,000 cubic feet per hour per million U.S. gallons." During this period the average reduction of free ammonia was about 20 per cent. and the reduction on kjeldahl nitrogen and in oxygen consumed was about

56 and 61 per cent. respectively. During the last four months of the year, when only 25,000 cubic feet of air per million U.S. gallons were used for five hours, the reduction in kjeldahl nitrogen was 56 per cent. That was due to the fact that a heavy brownish gray gelatinous film had been established on the slates. This film appeared to collect mechanically the suspended matter and a large part of the colloidal matter of the sewage. Messrs. Clark and Gage stated that "this tank produced a much better clarification of the sewage than did any of the clarification processes operated at this station, with the single exception of the one in which the sewage was precipitated with excessively large quantities of copperas and lime. The smallest average removal of total and organic matters in suspension was over 77 per cent." They estimated that with electric current at four cents per kw. hour the cost of aerating sewage would be about $1.85 per million U.S. gallons, using 25,000 cubic feet of air per hour.

The tank effluent was filtered through a perco-filter 10 feet deep; the quantity applied was gradually increased until 10 million U.S. gallons per acre per day was satisfactorily treated. The following table is copied from the report, and as in another part of the report it is stated that a filtrate containing 1.5 parts per 100,000 nitrates will be completely stable at all times, it will be observed that the filtrate in this case fulfilled that condition.

An interesting development is that described by Dr. Gilbert J. Fowler and Mr. E. Moore Mumford, of Manchester. These gentlemen found that by inoculating a tank effluent with an organism designated M_7, adding a small dose of ferric salt, about one grain per gallon, blowing air through the effluent for six hours and allowing it to stand for precipitation 6 hours, a very clear liquor could be obtained. This organism is to be found in nature in waters impregnated with iron and once its growth has been established there does not appear to be any difficulty in maintaining it.

The process which it is proposed to test at Baltimore is the "Acterated Sludge Process." Messrs. Ardern and Lockett, of Manchester, are its originators, and it has been tried on a large and practical scale at Salford, England, under the direction of Messrs. Duckworth and Melling.

Messrs. Ardern and Lockett had a large bottleful of sewage, through which air was blown for about five weeks, resulting in the complete nitrification of the sewage. The supernatant water was decanted and an equal amount of fresh sewage was added and air blown in, resulting in nitrification in a much less time. The procedure was repeated many times, until there was enough acterated sludge to inoculate fresh sewage in the proportion of one to four, or one to three, and an entirely satisfactory effluent was obtained after a few hours' aeration. The experiment was then made in casks placed outside and air was distributed through porous tiles, and after four hours' blowing in contact with acterated sludge 90 per cent. purification was effected, based on oxygen consumption and albumenoid ammonia or after six hours 92 per cent.; whilst the dissolved oxygen absorbed by shaken effluent in five days at 65° Fahr. was only 0.80 parts per 100,000, compared with two parts proposed under the British Royal Commission on Sewage Disposal report.

This process was tried on a still larger scale at Salford, where there was a tank already fitted with air pipes and available. There the purification attained after three hours' aeration was 90 per cent. on four hours' oxygen absorption and 76 per cent. on albumenoid ammonia, whilst the dissolved oxygen absorbed was only 0.46 parts per 100,000 when treating 90,000 gallons per 24 hours. Excess air, however, was used and the above results may not be obtained in ordinary operation, but there is a considerable margin permissible.

Acterated sludge process is applied at Wakefield, England, to difficult sewage, the large proportion of which is trade waste from factories. Similar tests are made at Henckley to treat dye water from factories with excellent results.

It will be observed that the underlying principle in the process of treating sewage by aeration is practically the same. In the slate beds it is the gelatinous film which must be established to obtain good results, in perco-filters it is the same, and in ordinary tanks it is the acterated sludge.

The sludge from the acterated sludge process is gelatinous in consistency, inoffensive and flocculent. It contains a higher percentage of fertilizing constituents than ordinary sewage sludge.

The quantity of air used was about 15 cubic feet per hour per square foot of tank area.

It is not to be anticipated that future developments shall constitute a reason for suspending the installation of new sewage disposal works, although the indications point to rather revolutionary changes in our ideas and methods. It would require a Jules Verne to imagine what the future works may be, and although we may revel in imagination we must abide our time. It seems probable that if aeration of sewage proves to be economical and practical on working scale, the existing works will be adaptable and perhaps may be made to treat a larger volume of sewage than is possible at present.

Average Rates and Quality of the Filtrate from the 10-foot Perco-filter at Lawrence Experiment Station, 1913.

Date	Quantity applied gallons per acre daily	Appearance		Ammonia			Kjeldahl Nitrogen	Chlorine	Nitrogen as		Oxygen Consumed	Bacteria per cubic centimetre
					Albumenoid							
		Turbidity	Color	Free	Total	In Solution			Nitrates	Nitrites		
January	1,000,000	0.5	.50	4.4700	.2437	.1747	.5433	11.87	0.26	.0463	1.45	795,000
February	1,854,000	0.3	.33	2.4300	.1250	.0970	.2525	8.75	1.57	.0219	1.02	855,000
March	3,714,000	0.4	.37	1.9574	.2540	.1380	.4760	11.68	1.58	.0094	1.66	90,300
April	5,037,000	0.4	.33	.6170	.1488	.1016	.3124	11.52	2.28	.0180	1.20	160,000
May	6,282,000	0.6	.32	1.7900	.1356	.0956	.2580	12.96	2.54	.0312	1.21	122,500
June	6,800,000	0.5	.33	2.3300	.1100	.0767	.2637	14.53	2.28	.0413	1.17	46,000
July	7,055,000	0.3	.34	.6620	.0724	.0608	.1938	14.60	2.22	.1152	1.01	150,000
August	8,117,000	1.4	.32	.7676	.1036	.0660	.2426	11.40	1.39	.0376	1.15	750,000
September ...	8,602,000	1.0	.33	1.0650	.1180	.0705	.2440	13.65	2.05	.0256	1.78	360,000
October	10,000,000	1.4	.31	1.1120	.1132	.0892	.2446	11.32	1.42	.0460	1.07	55,000
November ...	9,869,000	1.3	.36	1.5600	.2030	.0970	.5098	12.30	1.33	.0285	1.91	24,000
Average ...	6,212,000	0.7	.35	1.7055	.1479	.0970	.3219	12.23	1.72	.0383	1.33	309,800

IMPROVED METHOD OF FILTER CLEANING.

A THOROUGH study of filter cleaning methods in use by the Philadelphia Bureau of Water was made by Mr. Sanford E. Thompson, and the recommendations he made for improving the efficiency of that work, together with the results obtained, contain many interesting points. A review of his study of the problem forms the basis of a paper read by him recently before the American Society of Mechanical Engineers.

The particular operation studied was the removing and cleaning of filter sand. The city has five large filtration plants consisting of covered reservoirs operated by slow sand filtration. In the filtration plant first handled by the method recommended by Mr. Thompson, there are sixty-five filters which are cleaned by about 128 men. Each filter is about 140 feet wide by 250 feet long and is built with groined arch bottom and roof, the supporting columns being about 16 feet apart on centres. The Nichols method of washing is used in this plant, sand being shovelled into an ejector, carried by water to a separator and the clean sand returned by a hose to the bed, where it is properly distributed and levelled.

Four washing gangs are required for each filter bed, the outside gangs having two and a half bays each and the inside gangs having two bays to clean. In each gang there are three shovellers to a hopper, two men shovelling at a time while one rests. Each man shovels forty minutes and then rests twenty minutes. A fourth man takes care of the hose from the separator, distributing the clean sand to the bed. A fifth man, recently introduced, working with two gangs, spades up the hard sand that has been uncovered before the replacing of the washed sand.

The object was to lay out the work of each gang so as to increase the effectiveness of the plant and provide a definite task to be accomplished in a day. The results of the plan which is being put into operation are as follow:

Rotation of cleaning the filters is planned in advance by well defined rule.

A definite area of sand to clean is assigned to each gang, this area depending upon the depth of cleaning necessary.

This setting of tasks has increased the output of each gang 15 per cent., and this should be further increased to at least 25 per cent.

Accurate records are kept showing the time consumed by each gang.

Cost accounts, as well as payroll, are made up from the time tickets furnished to the men.

Gang leaders are required to pay closer attention to their duties.

Improved apparatus and machinery are under consideration.

Methods of determining depths of sand to clean are being standardized.

At the beginning, studies were made of the men and methods employed to see whether the manner of handling the work could be improved. Time studies were made to determine the unit times for each individual operation, so that the tasks could be figured accurately in advance.

Time studies were made, by the aid of a stop watch, of the labor operation in the beds, such as shovelling dirty sand to hopper, cleaning up around hopper, moving hopper, moving separator and moving track. These times for individual operations were then converted for direct use into the time per cubic yard for one inch of depth. It was found that moving the hopper required a unit time of 0.2 of a minute, moving separator 0.5 of a minute, moving hopper hose .25 of a minute, moving track .83 of a minute, waiting for hopper to empty .42 minute, moving pressure hose 1.8 minutes, shovelling to hopper 6.32 minutes per cubic yard per one inch depth, and additional necessary rest .12 minute per cubic yard. Having determined the unit times, the area of surface that should be shovelled by each gang was figured and the point to which they were supposed to go in a day's work was marked with a flag. (In order to fix this, it is necessary to determine in advance by test holes the depth which should be cleaned.) Curves have been plotted giving distances a gang should cover for each depth of cleaning. Calculations of the area the gang should shovel were made without informing the gang, and for two days the amount covered by each gang was from 10.5 to 31.5 per cent. less than this. A task was then set for one gang, and they readily accomplished this task; after which a daily task was set for each of the gangs.

After this new method had been employed for some time, it was compared with the old by taking an average of twenty-seven cleanings at random from a period of one and a half years. These showed an average rate of 6.3 cubic yards shovelled per day per gang. An average of fifty-five cleanings after task work was started gave 7.2 cubic yards, an increase of nearly 15 per cent. The figured rate, however, was 8.4 cubic yards, or more than double this increase, and it is believed that this could be reached with first-class supervision; and that 50 per cent. increase could be obtained if it were possible to pay a money bonus. A plan considered as a partial incentive is to keep a record card for each man, showing his output and thus indicating his relative rank as a worker, which rank would influence his retention when work was slack, or his promotion to a higher position.

The studies indicated a number of changes desirable in the apparatus and methods of handling it. It was found that the lines of piping for the water used in the cleaning operation were poorly arranged, requiring too long lengths of hose in certain cases; while in other cases pipe lines had to be moved from bed to bed during cleaning. It has also been shown that a mechanical washing contrivance probably can be devised which will reduce the cost of cleaning. It was also found that the hoppers and separators were of such size that they would not handle just the amount which the gang could readily shovel; the hopper, for instance, having an output slightly greater than could be shovelled into it by one man, but not nearly as much as could be shovelled by two.

SLAG FOR CONCRETE AGGREGATE.

Comparative tests of trap rock and furnace slag as aggregate for concrete were recently made by Prof. H. Perrine, of Columbia University. The tests consisted of making compression tests on 8-inch cylinders of concrete, mixed one part cement, 2 parts sand and 4 parts of either Palisades trap rock or slag furnished by the National Slag Company. The rock was separated into ¾-inch, ½-inch, ¼-inch and "dust," and then artificially recombined so that the grading was identical with that of the slag, which was used as received. The materials were proportioned by volume and mixed in a Blystone batch mixer.

When the cylinders were 28 days old they were tested to rupture. The trap concrete showed ultimate strength of 1,769 to 2,120 lbs. per sq. in., averaging 1,975.5 lbs.; while the slag concrete showed 2,275 to 2,750 lbs., with an average of 2,465.5 lbs. The former weighed on the average 154.5 lbs. per cubic foot, and the latter 140.6 lbs.

HYDRO-ELECTRIC POWER DEVELOPMENT AT JORDAN RIVER.

THE Jordan River power development, about 36 miles West of Victoria, is owned and operated by the Vancouver Island Power Company, a subsidiary of the British Columbia Electric Railway Company, which is the principal public service corporation of Victoria. This development is the subject of a very complete and comprehensive paper presented to the Canadian Society of Civil Engineers on March 4th, by Mr. Charles A. Lee. As the author states, articles have appeared in engineering journals during the past two years describing parts of this development and in this connection the reader is referred to two extensive articles which appeared in *The Canadian Engineer* for November 28th and December 5th, 1912. These articles gave a full description of the whole undertaking up to that time. It has remained for Mr. Lee, however, to present a description of the development in its complete form, as important additions have been made to meet the steadily increasing demand for electrical energy made upon it.

Preliminary surveys began in 1907 and were completed in 1908. In the following year construction began

Fig. 1.—General View of the Jordan River Dam During Construction.

and the original installation, as described in the above article, was completed in 1912. Demand for power was so great, however, that a second unit was installed in the fall of 1912, marking the ultimate capacity of the initial development. Further additions to the development necessitated an increase of storage capacity, and enlargement of pipe line, and a new power house, etc. The storage increase was obtained by the construction of the Jordan River dam completed in the spring of 1913, followed by flume and pressure pipe line construction and by additions to the power buildings. The whole was completed in October, 1914. We extract the following from Mr. Lee's paper respecting these recent additions in order to supplement the information which appeared in this journal in 1912, and to bring our treatment of the subject up to date.

Briefly, the power house is located just above high tide at the mouth of the river. Steel pressure pipe lines about 9,300 ft. long convey the water to the wheels from the forebay reservoir, which is formed by two earthfill dams, giving a static head of 1,145 ft. at the power house. Water is conveyed to the reservoir in a wooden flume for a distance of 5.3 miles. A small dam in Alligator Creek diverts the water therein to a flume joining the main flume about a mile below the main diversion point on Jordan River. Wye Creek joins the river above the diversion dam, and Bear Creek also joins it about 3½ miles above the dam. The Bear Creek storage dam lies about a mile above the juncture of Bear Creek and Jordan River. As

Fig. 2.—Looking Down-stream, Jordan River Dam, During Construction.

stated above, a full description of the layout and of the design and construction of the Bear Creek dam and of the Wye Creek diversion dam was presented in our previous article upon the subject.

The Jordan River Dam.—When the provision of greater storage was found necessary, it was decided to develop a reservoir at the diversion point. A hollow reinforced concrete dam of the Ambursen type was chosen and its construction commenced in August, 1912. It backs the water up into the two creek valleys, forming a lake with an area of 398 acres at the spillway level and with a capacity above the outlet gates of 612,000,000 cubic feet.

Fig. 3.—Jordan River Dam Completed.

The crest of the dam is 891 ft. in length, of which 130 ft. is earth embankment with a concrete core-wall. The spillway is located well toward the east end of the dam and is 305 ft. long with the crest 8 ft. below the top of the dam, providing a flood discharge capacity of 23,000 cubic feet per second. The curved crest and rollway apron discharge the water clear of the toe of the dam and into a natural channel across the flat. This channel joins the river about 200 feet below the dam. The extreme height of the dam is 126 feet from the deepest part of the buttress foundations in the river bed to the crest.

The foundations were not stripped to bed rock except those portions to be occupied by the buttresses and cut-off trench. The latter was excavated along the upstream toe of the dam and varied in depth from 3 to 12 ft. to assure watertightness. The dam has a reinforced concrete face inclined at an angle of 45 degrees and supported on concrete buttresses spaced 18 ft. apart across its whole length. The buttresses are 12 inches thick at the top and increase, by steps or lifts 12 feet high, to 42 inches in thickness at the bottom of the highest buttress. The upstream edge is built on a slope of 1 to 1, and the downstream edge has a batter of 1 to 4 to a point 18 feet below the crest, from which point it is vertical to the crest. Just back of the upstream edge a heavy reinforced haunch or shoulder is built on either side of the buttress and the decks are supported on these haunches. The buttress projects beyond the haunches a distance equal to the thickness of the deck, and a bonding groove or key is cast in this projection. No vertical reinforcement is used in the buttresses excepting along the downstream edge and in the haunches, which are heavily reinforced to carry the decks. Horizontal reinforcement is used along the top and bottom of each of the 12-foot lifts or steps. Horizontal columns or tie beams, which are reinforced top and bottom, connect the buttresses at various elevations and give them lateral support. The reinforcement in these beams is continuous through each three consecutive buttresses, but is not carried continuously through the dam on account of possible strains set up by expansion and contraction.

The decks are designed as simple beams uniformly loaded. The thickness varies from 15 inches at the crest to 55 inches at the cut-off wall in the deepest part of the dam. The horizontal reinforcement consists of ⅞-inch square corrigated bars on 4-inch centres up to a point 45 feet below the crest of the dam, and above that point on 4½-inch centres. The vertical reinforcement consists of six ⅝-inch square corrigated bars in each bay or deck slab.

The crest of the dam is 6 feet wide. The spillway crest is 4 feet wide. The apron of the latter is built on an ogee curve and is designed for a depth of 7½ feet of water over the crest of the spillway.

The earth fill at the east end of the dam is about 130 feet in length reinforced with a concrete corewall 3 feet thick at the bottom, tapering to 1½ feet at the top, founded on bed rock and reinforced horizontally and vertically. The maximum height of the wall is 44 feet, 28 feet of which is below the original ground surface. The crest width of the fill is 10 feet and the slopes are 2 : 1.

The outlet occupies the space of two buttresses near the west end. The bottom of the gates is 93 feet below the crest of the dam. There are two openings in each face of the dam and two sets of two gates control the flow of water. Each set consists of an emergency gate set at an angle of 45°, or parallel to the deck of the dam, and behind this gate, but in the same chamber, a service gate set vertically. All gates are identical in design and size, and have a clear opening 5 ft. 6 in. high by 3 ft. 6 in. wide. The gate stems pass through the roof of the gate chamber in oil-filled pipes with a packing gland on each end. The operating stands are placed on a concrete floor about 3 feet above the roof of the chamber. These are geared, roller bearing stands, and are so designed that one man can operate the gates under the maximum head. The gates and operating gears were supplied by the Coldwell-Wilcox Company.

The outlet gates discharge directly into a settling basin, and from here the water enters the flume. Sand gates are provided for discharging any silt and sand which may settle in the basin.

The outlet openings are protected by two trash racks made up of ¾-inch by 3-inch bars placed on edge on 5-inch centres. These rack bars are mounted on channel iron frames which are supported on flanged wheels and run on rails up and down the face of the dam. The rails are laid at two different levels on concrete projections or walls on the face of the dam, so that one rack passes under the other one. By this arrangement either rack can be lowered in front of the outlet openings while the other one is hoisted to the top of the dam to be cleaned. The racks are lifted by means of ¾-inch crane chains which pass over suitable hand gears on the dam crest and are attached to counterweights inside the dam. Each rack has approximately 450 sq. ft. area.

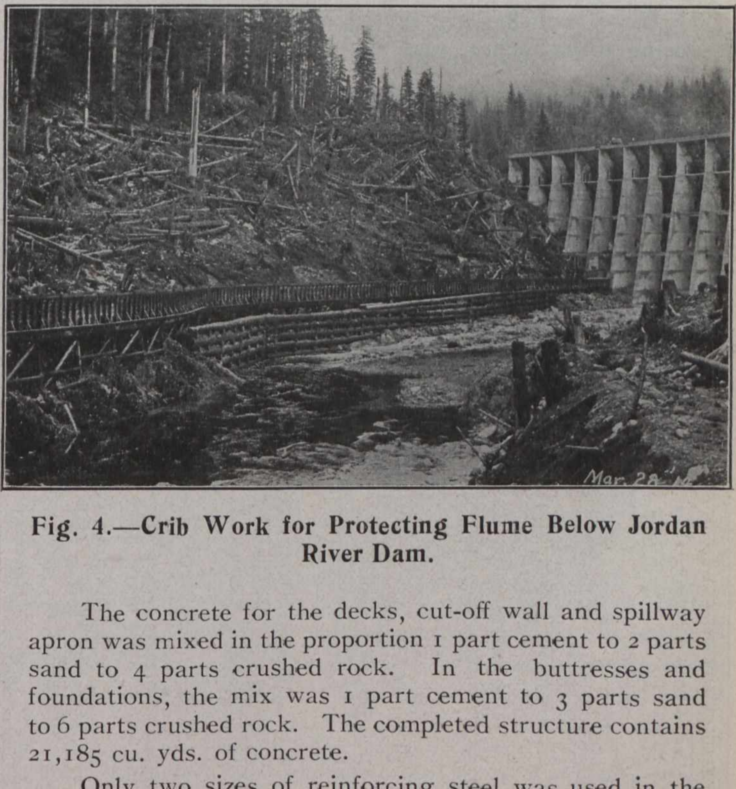


Fig. 4.—Crib Work for Protecting Flume Below Jordan River Dam.

The concrete for the decks, cut-off wall and spillway apron was mixed in the proportion 1 part cement to 2 parts sand to 4 parts crushed rock. In the buttresses and foundations, the mix was 1 part cement to 3 parts sand to 6 parts crushed rock. The completed structure contains 21,185 cu. yds. of concrete.

Only two sizes of reinforcing steel was used in the entire dam, these being as mentioned above. The total weight of steel used in the dam was 380 tons.

Unit costs were as follows: Quarrying and crushing the rock and sand used in the concrete, $1.86 per cu. yd.; mixing, placing and puddling the concrete, $2.52 per cu. yd. The cement and reinforcing steel cost $5.23 per cu. yd. Form work, including first cost of moveable forms and the moving and stripping of all form work, $3.45 per cu. yd. of concrete. The total cost of the concrete in the completed structure was $14.01 per cu. yd.

Power House Addition.—The addition, started in spring of 1913 and finished in September, 1914, was constructed as a building separate from the old power house, and consists of a generator room 120 x 47 ft., and behind this a high-tension switching room 130 x 27 ft. The floor of the new building is 5 ft. higher than that of the old, which was only 7½ ft. above high tide. The building and the machinery rest upon 102 concrete piles cast 12 inches diameter and varying in length from 45 to 75 ft., the average length being 56½ ft. The floor of the basement is a reinforced concrete slab 8 inches thick, and is increased over the piles supporting the machinery to 2 ft. thick. The walls are 12 inches thick up to main floor level and 8 inches thick above. They are stiffened by pilasters at 13½ ft. centres. The roof is a reinforced concrete slab 3½ inches thick and supported on steel roof trusses and I-beam purlins.

Space is provided in the new building for two generating units, only one of which has been installed. This unit, known as Unit No. 3 of the development, is an 8,000-kw., 2,200-volt, 60-cycle, 3-phase C.G.E. machine driven at 400 r.p.m. by two Doble tangential water wheels, one mounted on each end of the shaft and overhanging the bearings. They are rated at 13,000 h.p. The shaft is a hollow nickel steel forging 16 inches in diameter in the bearings, which are 60 inches long.

Water is supplied to the wheels through a flanged cast-steel wye, bolted to the pressure pipe. It is 48 inches in diameter at the entrance and 34 inches at the branches. To these branches are bolted cast-steel taper pipes, reducing to 24 inches, and to these are bolted the 24-inch single disc, steel-body gate valves. These gate valves are operated by small, reversible water wheels, which are mounted on brackets on the yokes of the valves. The water motors operate the bronze nut on the rising stem of the valve through a system of spur and bevel gearing,

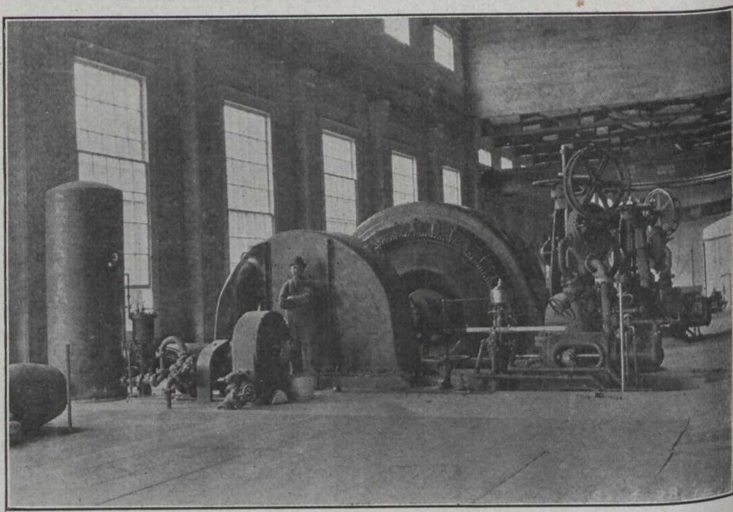


Fig. 6.—13,000-h.p. Generating Uuit.

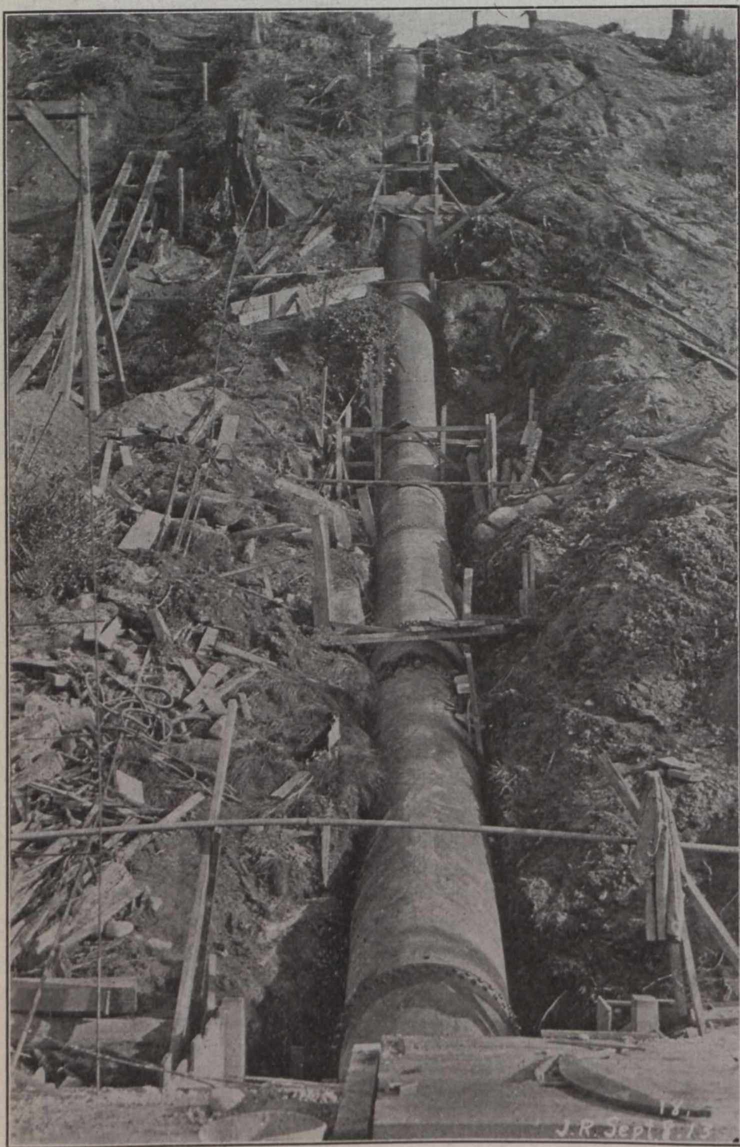


Fig. 5.—Pipe Line to No. 3 Unit. View from Power House.

and provide a dependable means for opening or closing the valves. Water is supplied to the wheels through short pipes connected to the hood of the valve, and an automatic device is provided which prevents over-running. The cast-steel nozzle bodies, with main and relief nozzles, are bolted directly to the gate valves.

Each wheel is provided with an entirely separate direct-motion, oil-operated, relay type governor, connected with which is a special hand control.

No. 3 exciter unit is a 200-kw. generator direct connected to a 200-h.p. water wheel and a 300-h.p., 2,200-volt induction motor.

It should be stated that Unit No. 2 is identical in size and design with No. 1, described in the article previously referred to. Each is a 4,000-kw., 3,200-volt, A.C.B. generator driven at 400 r.p.m. by a single Doble tangential wheel of 6,000 h.p.

The unit cost of the Jordan River power plant, including the power house building and all hydraulic and electrical equipment, is $20.60 per h.p. capacity. The present capacity is 25,000 h.p., but provision has been made for the addition of one 13,000-h.p. unit, and when this machinery is installed the unit cost will be reduced to about $17 per h.p.

The additions to the development, referred to in this article, were constructed under the direction of Mr. G. R. G. Conway, M.Can.Soc.C.E., chief engineer of the company. Mr. Lee had charge of construction during this period.

FREIGHT RATES ON PULPWOOD.

United States shippers and consumers of pulpwood petitioned the Interstate Commerce Commission recently against the present joint through rates published by Canadian railroads, and concurred in by roads in the United States. The Commission could not meet with their requests, however, claiming that since the United States lines merely concurred in the rates it could do nothing more than order them to desist from such concurrence, leaving the old combination of rates to and from border points in effect. The rates which caused the United States pulp men to approach the Commission in this respect were found quite reasonable by the Board of Railway Commissioners for Canada.

STUDIES IN ROAD CONSTRUCTION.

(Continued from last Issue.)

STONE ROADS (WATERBOUND MACADAM).

By Hugh A. Lumsden, B.Sc., A.M.Can.Soc.C.E., assistant engineer Ontario Office of Public Highways.

The author, after pointing out that traffic was the consideration which governed chiefly the selection of a particular type of road, and showing the wide variety of types to be found in Canada, stated that over 3,000 miles of waterbound macadam had been laid since the introduction of the county road system in 1901. Some of them, despite lack of maintenance, were in good condition, though carrying a heavy daily traffic after five or six years of use. Others had failed and had thereby provided a great reason why there should be fewer failures in the future. He dealt with the construction of a waterbound macadam road under the following headings: Its necessity; drainage; grading and surfacing; stoning and maintenance. It was clearly pointed out that there is a certain field for such a road; that while each road was a problem in itself, it could be generally stated that the construction of a waterbound macadam road should be most seriously considered when a traffic census showed that from 100 to 200 vehicles of all classes used the road each day, and that this rule would obtain when stone had to be shipped in for use. Where local material was available, traffic much less than 100 vehicles per day, or where a census showed that 200 vehicles per day would be greatly increased after the improvement was made, were variations to the above rule subject to careful consideration.

Relating to drainage, the author emphasized it as the most important item in road construction, as had already been brought out in previous papers. Adequate drainage might be assured in the following way:

The desirable width for a road to-day is from 24 to 27 feet between ditches. As to the dimensions of these ditches, the slopes should be 1:1, the bottom generally the width of an ordinary small drag-scraper and at a depth of at least a foot below the roadbed. The depth and dimensions of the ditches will, of course, vary with the soil and with the local conditions. It is where heavy clay soils are encountered that drainage becomes a particularly difficult and often expensive problem. The solution is very generally found by the use of properly constructed tile drains. The author believed tile laid on both sides of the road to be generally preferable to the single drain laid under the centre of the road, and well worth the additional cost. The tile, never less than 4 in. in diameter, should be laid at a depth of three feet below the bottom of the ditches; it should be well packed about with stone and filled up over it with stone for about 2½ feet. Provision must be made to ensure a good run-off for the tile, for which purpose a drainage award is often the most satisfactory way to have the work properly completed and the cost equally proportioned. While the cost of a tile drain varies greatly, a fair average for the laying and backfilling of same, including cost of tile, would run about 5c. to 10c. per foot for 6-in. tile.

Grading and Surfacing.—To properly carry out such work, road machinery is an absolute necessity. Several ploughs and scrapers, very often wagons, and always a roller of at least ten tons are necessities for the proper construction of the subgrade.

The extent to which hills should be cut down and hollows filled up is largely an engineering problem, which varies with each particular road. The lowest possible grade, while always most desirable, may very often not be the most economic construction. Long grades of even as great as 4% or 5% should not be subjects of great concern; above this, however, the increased cost of haul mounts rapidly.

The most desirable width for the subgrade is from 24 to 27 feet between ditches; the shoulders should have a slope of 1½ inches to the foot. For a double-track road 18 feet of stone is considered the most economical width.

Not till the road has been carefully prepared and thoroughly rolled is it ready for the stone to be placed.

The question as to the most suitable sizes of stone for use on the roads is one of great importance and one on which considerable diversity of opinion seems to exist. Experience seems to show that for stone roads of limestone the use of a screen containing only two sizes or rings, namely, 1½-in. and 3-in. holes, is best suitable. From such a screen we can then take out: (*a*) Tailings or those crushed stones refused by the 3-in. rings; (*b*) stone rejected by 1½-in. rings but passing through the 3-in. rings; (*c*) screenings, stone dust and all passing through a 1½-in. ring.

By reason of the fact that trap rock is so very much harder than limestone, the proportion being about 7:4, it is desirable that when trap or granite rock is being used for the road metal a screen of smaller rings be used. Rings of 2½-in. and of 1-in. are suitable.

We shall then obtain (*a*) tailings stone rejected by the 2½-in. rings; (*b*) stone passing a 2½-in. screen, rejected by a 1-in. ring; (*c*) screenings and stone passing a 1-in. ring.

Construction.—The tailings should first be placed and spread over the centre of the road; these constitute but a very small part of the whole. The next sized stone (*b*) is then placed on the road to a sufficient depth to make with the tailings, after rolling, the desired thickness. This thickness runs from 6 in. to 12 in., depending on the sort of soil on which the road is being built; 8 to 9 inches generally makes a very suitable thickness. The stone should then be rolled to a level surface. In this operation the skill and ability of the roller operator is of the greatest importance. It is his duty to instruct his assistant to fill up all small depressions, and to see that on the completion of rolling the road presents a smooth and even surface throughout.

A light coating of screenings (*c*) is next placed and the road thoroughly watered and rolled, the purpose being to fill up the interstices in the coarser stone with those of smaller size and at the same time flush the screenings into the voids. Much rolling and watering will be required to accomplish this. The road should be wet and rolled till it puddles on the surface showing the voids to be substantially filled. Once this is done only a very light covering of screenings should remain over the road surface. Thorough rolling of the completed surface and shoulders completes the work.

To what extent does rolling compact broken stone? In loose broken stone the voids are about 40%. Were this stone on an absolutely solid foundation, it has been proved that 6 in. of stone cannot be compacted to 4 in. by rolling; *i.e.*, it is impossible to reduce the voids to 6%. But 7.38 in. of trap rock after rolling was compacted to 6 in., which represents about the usual reduction in thickness of stone after rolling. It is the pressing of the stone into the subgrade which tends to give an idea that the compression is greater than it actually is.

The author then commented as follows on familiar faults and difficulties of construction: One dollar of drainage may often be of more permanent value to any road than five dollars spent on the stone. The better the drainage the less may be the required thickness of stone, and the danger from frost is small. The wetting of stone after the first course of stone is on the road is not advisable; on the contrary, it merely tends to make the subgrade muddy. In bringing the second course or the screenings on to the road, the previous course, where available, should not be teamed over. For the teaming over the unfinished road will tend to form ruts which will very probably be filled up by the screenings. Occurring, as these ruts will, just where the future line of travel will be greatest, the road has not then the substance to withstand the traffic just where it is most required, and speedy rutting is the result. When distributing the stone and screenings, piles should be left alongside the road every half mile to provide for future maintenance. Watering carts should have wide wheels to avoid rutting the unfinished surface.

Should one varied grade of stone only be used which means simply crushed stone covered by screenings? This is a big question on which there is diversity of opinion. While it is true that the voids may thus be somewhat better filled, on the other hand it is claimed the larger stones are thus as likely to be near the top as the bottom and will tend to unravel from the road surface quicker than the uniform stones of smaller size.

The cost of macadam roads in Michigan averages just under $4,500 per mile. In Halton County this year the average cost is about $2,900 per mile, including 27-in. pipe of iron or concrete and three concrete culverts. In Peel County, $4,500 for imported stone and $2,900 for local stone per mile.

As regards costs every road is a problem in itself depending on local conditions, and averages are therefore of little use to one desiring to know the probable cost of a proposed road.

To illustrate this in Halton County: In certain parts of Nassagawega Township, stone roads were built this year at a cost of about $1,500 per mile, the stone itself being delivered on the road for less than 50c. per cubic yard and the contractor made a little better than wages; in this case, quarries were directly alongside the road. On the other hand, in Trafalgar township where no local stone was available, crushed stone had to be shipped in from Hagersville at a cost of $1.35 per yard and an additional 50c. or thereabouts required to haul stone to the road. The average cost of roads built in this township was $4,300 per mile.

Maintenance.—If a railroad constructs a line, however short, it immediately assigns a gang of from 4 to 6 men to look after the upkeep of each section of the road, about 7 miles to each section. Twenty or thirty trains a day might be a large traffic over this road. Why, then, do we continue to construct roads over which as many as two or three hundred vehicles may pass daily and afterwards neglect them entirely until re-building is necessary. Considering maintenance from an economic standpoint, surely we should follow somewhat the railway practice and adopt a patrol system, immediately following the construction of a road.

GRAVEL ROADS. By J. A. P. Marshall, B.A.Sc., assistant engineer, Ontario Office of Public Highways.

Gravel was defined and its great use and value as a surface covering for county roads in Ontario, brought out. The author denoted its wide variation in quality and emphasized that hardness, toughness and durability were essential, combined with the property of binding into a solid mass under traffic. The function of the binding material, generally clay, which commonly occurs in gravel and which, to prevent mud in bad weather, should be less than 20%, or, iron oxide, lime or finely divided silica was described. It was shown that just enough binder to consolidate the particles of gravel should be used.

The occurrence and nature of gravel deposits throughout the province were described.

Concerning the construction of gravel roads the author stated that the subgrade on which the gravel is to rest should receive just as careful attention as if it were being prepared for a broken stone surface. Thorough drainage and an adequate foundation for the road must be provided. The subgrade must be so prepared that it offers a firm surface on which the gravel top may be constructed. The crown should be made one inch to the foot.

Surface Method of Construction.—The subgrade is brought to the proper shape by use of the grader and firmly compacted. The gravel is dumped on the roadbed and smoothed out to the desired shape, the larger stone being raked over in the bottom of the road. The width and depth of the gravel surfacing necessary will depend upon the traffic conditions. A width of 8 feet is a fair average for country roads, the earth grade to be from 18 to 24 feet wide between the inside edges of ditches. A depth of 8 in. to 10 in. is a good rule at the centre. To leave gravel in an irregular ridge as it falls from the wagons is dangerous and wasteful. Care should be taken in alignment and the roller should be used in compacting the surface. The surface method is the one most generally used in Ontario.

Trench Method.—A trench is constructed in the subgrade of the same width as the gravel surfacing. The subgrade is otherwise prepared in the usual manner so as to furnish a firm support for the gravel surfacing. The trench is obtained by building the portion of the roadbed prepared for the gravel below the sides an amount equal to the thickness of the gravel surface. On embankments the distance from the edge of the gravel surface to the edge of the slope of the bank should be at least 3 feet. The final compaction of these shoulders cannot be accomplished until after the gravel surface has been completed. When finished, the shoulders should have the same slope as the rest of the gravel surface so that a continuous slope is obtained from the centre of the road to the outside edge of the shoulder.

This method is not used to any extent in Ontario at the present time, but it might be advantageous in roads nearing towns and cities where the surface would probably be treated with a bituminous binder or tar product.

Gravel containing a mass of large stones and boulders should be treated as rock, and put through a crusher. A rotary screen attached to the crusher is desirable to separate the crushed stone into coarse and fine grades. Where there is an excess of clay, earthy matter or sand, a rotary screen is especially useful in removing it. When gravel is not screened, very much may be accomplished by care in selecting and taking it from the pit.

Gravel should be bought not by the load, but by the pit or by the acre and should be available at all times. Especial care should be taken by councils to see that prior to the performance of road work, the pit is stripped and the gravel treated if necessary.

Cost of Gravel Roads.—The cost of gravel roads throughout the province varies considerably. In some counties gravel will cost 60c. per yard, delivered on the road and in others $1.50. A fair average price is $1,200 per mile. In localities where gravel is not within easy hauling distance, however, it will be advisable to consider the use of other metal; since gravel is not found in the immediate locality, and therefore expensive in transportation, will frequently be found less economical in final cost than a better, though higher priced, stone.

It may take several months before a gravel surface is thoroughly compacted, no matter how well it may have been rolled during construction. During this period careful attention should be given to the road, and the ruts and hollows should be patched as they are formed. In times of wet weather or of frost, a gravel surface will be soft and rut very easily. As in the case of earth roads, the road drag is one of the best tools with which to maintain a gravel road. Where a road grader would unnecessarily disturb the surface, the road drag serves to simply smooth up the road, fill in the hollows and put just enough of the material towards the centre to maintain the crown.

An essential part of the maintenance work is to keep ditches, drains and culverts clear to provide for the removal of the water which falls onto the road.

In conclusion, perhaps the greatest advantage peculiar to gravel comes from the presence of the binder which possesses the power of quickly reconsolidating the material under traffic, even after the first bond has been broken, thus reducing repairs and maintenance to a comparative minimum.

MAINTENANCE AND REPAIR OF HIGHWAYS.

By Charles Talbot, County Engineer of Middlesex.

Maintenance was referred to in this paper as applying to the up-keep of a road after construction, so as to continuously meet the requirements of the traffic to which it is subjected. The author pointed out that no attempt had been made in this respect toward the maintenance of public highways in the rural municipalities of Ontario. Almost without exception the new or reconstructed roads were left to take care of themselves after construction until they were in such a condition as to require rebuilding, before it was considered advisable, or just to other sections of the municipality, to spend further money upon them. The fallacy of such a practice was strongly pointed out. The author observed that the Legislature, while encouraging road construction by the offer of one-third of the cost of the work, had not encouraged maintenance, other than calling attention to the immediate and constant necessity of taking care of the highway after construction. No financial aid was given towards maintenance and the representatives invariably considered it more advantageous to apply the whole of their available funds to construction. A third reason was that the man in charge of maintenance would be working continuously on a job where neither the necessity of the work nor the result produced was apparent to the casual observer.

The author referred to that portion of the Municipal Act requiring townships to construct and maintain all roads within their municipality, and pointed out that townships adjacent to large cities and shipping stations had an unreasonable burden to bear in this respect. It was observed that the Highway Commission, recognizing this condition, had made recommendations to adjust the matter, bringing relief to such townships. The author suggested that a step farther might well be taken, requiring the county councils to assume their share of the responsibility.

The Highway Commission had also recognized the need of the Legislature contributing toward the maintenance of highways, and if their recommendations are adopted they will remove the second reason for lack of maintenance, as mentioned above.

To overcome the third reason, the author commended the plan suggesting that the management of the road be removed from direct control of local influence, as provided for in the proposed Act.

It was recommended that every county having supervision of highways should have a permanent officer in charge of the work, with his duties clearly set forth, and with complete supervision of all work, including absolute control of all employees. The county roads should be divided into sections of from five to ten miles, according to traffic, and a man placed in charge of each section, with log drag or heavy grader. Under the advice and direction of the engineer, he would be responsible for the maintenance of his section. He should have supervision of all contracts and of all day labor work and should at all times keep proper time sheets and records of material. This road foreman should be given power to employ men and teams as required, and should be responsible for the care of all road machinery.

The author went on to state that the average rural road once properly graded, drained and metalled requires comparatively little attention to keep it in excellent repair. The little it does require must be applied promptly and before the necessity is apparent to an uninterested person. The greater part of the actual labor necessary will consist of the timely use of the road grader or drag, the delivering and placing of the necessary gravel or stone to maintain the crown, repairing tile and open drains, cutting weeds and shovelling snow. All of this work, except the operating of the grader and the placing of the metal can be attended to by ordinary labor and teamsters, which are at all times available in such numbers that the work can be quickly done and at a very moderate cost if properly directed by a competent foreman.

With a superintendent and foreman working in harmony with the council there is direct and efficient control of every mile of road either for constructing a new road, reconstruction of an old road or for maintenance of one already completed.

On a stone road, ruts will develop on certain sections, while other portions remain solid and firm. Except for a slight wearing of the stone and the removal of a part of the binder exposing the surface of the larger stone, little effect is noticeable upon the road for a few years. What are the conditions that produce the different results?

Except where very heavy travel exists it will be found where a stone road ruts, the rut is not caused by a wearing of the stone, but by the weight of the wheel traffic forcing the stone into the grade beneath. From this, one must conclude the foundation was defective. Sub-soils composed of a rich black loam, although reasonably well drained, will still lack the carrying capacity of the firmer soil and hence the rut. Firmer soils with insufficient drainage will produce the same results.

If the trouble is caused by an open and yielding sub-soil the ruts should be cleaned, tar painted and stone tamped to position, repeating the operation as often as necessary to form a firm foundation. If the original depth of stone is obviously too thin, a second coat should be applied and finished with the steam roller.

If the trouble is with the drainage, sufficient drains must be placed in the road. If the sides are previously tiled and drains in working order, as they should be in every case where clay or loam or other damp sub-soil exists, it will almost invariably pay to open the centre of the road and construct a tile drain from 2 ft. to 2 ft. 6 in.

below the surface and fill the trench with coarse gravel, brick bats or stone; then replace the stone, add sufficient new material and roll to a finish. This is somewhat expensive, but will, without fail, give results. If the repair is necessitated by the wearing of the surface by the traffic the surface should be properly cleaned and scarified and refinished, adding sufficient stone to restore the crown to its original condition. Small depressions developing in the road should be filled before they are deep enough to retain water and only sufficient stone used to level the defective spot.

In a gravel road will exist the same necessity for a firm sub-soil and a proper drainage. Almost invariably, however, where gravel is used the price will not be excessive and it is placed in larger quantities on the road, consequently the rutting is not so usual as in stone.

Then, again, in most grades of gravel used, a few wet days will loosen the surface so the grader can readily level the road and fill the ruts. This is impossible with the stone road; consequently the gravel road is much more easily kept level and smooth during the season. Springtime produces a condition for a short period that is objectionable and that requires careful handling, but by adding sufficient gravel from year to year to maintain the crown and by repeated use of the grader as required, the gravel road will answer every requirement of a mixed traffic upon a road that does not exceed 350 vehicles per day.

QUARRYING AND EXPLOSIVES. By R. M. Smith, B.Sc., assistant engineer, Ontario Office of Public Highways.

These two subjects were dealt with in separate papers and contained a great deal of useful information for the road builder who had to do with quarrying in the course of his work. The subject of explosives was given thorough treatment, the various types and their respective characteristics and uses being fully explained. Canadian use of explosives involves black powder (gun powder and blasting powder) chiefly, and a few of the ordinary varieties of dynamite. There appears to be rather a disinclination to depart from them in favor of newer types.

The paper classifies explosives and briefly explains their different compositions, blasting effects, rates of detonation and burning, and efficiency in respect to desirability of a shattering or a heaving effect. Instructions are given for keeping and using the more commonly used explosive compounds. The requirements which the product should fulfil are outlined. Cautionary remarks are made respecting the storing and handling of dynamite and of thawing it where necessity demands.

The author then gives directions for placing charges of both dynamite and black blasting powder and for the manipulation of fuses and caps. The following is a brief extract:—

The engineer and quarryman are more concerned with the selection of an explosive to get the most efficient results. For rock in highway work the low freezing dynamites are probably the best adapted. Where a strong smashing action is desired the nitroglycerine grades can be used, and where more of a spreading and heaving action is necessary blasting powder will give the best results.

Mistakes are often made in regard to producing rock for macadam. If the rock is hard and flinty a high explosive breaks it into pieces or spalls, full of flaws, and the additional fracturing in a stone crusher renders it unfit for heavy traffic because it grinds down too quickly. A milder explosive causes fewer small fractures and the life of the macadam is much prolonged.

General Rules.—(1) Do not allow cartridges with caps attached to be scattered around.

(2) Dynamite should not be packed in same box with caps or fuse, and they should never be brought together until ready to use.

(3) Avoid bringing matches or other easily combustible substance near an explosive.

(4) Avoid the use of hard or rigid tools. Copper is the only metal which may be used about explosives.

(5) Use only quantity of explosive necessary for work in hand. Keep main supply well removed from vicinity of work, well protected from fire or shock and from being handled by unauthorized persons.

(6) In transporting nitroglycerine, guncotton (dry), dynamite and gelatine it is necessary to have an elastic bed of hay, straw or excelsior to deaden the shock in travelling.

(7) Never prepare a dynamite primer near nitroglycerine compounds, or explosive gelatine.

(8) Before firing, warning should be given to all persons.

(9) In case of misfire, do not make haste to investigate cause. Wait at least 20 minutes; then, when satisfied that no explosion will take place, remove as much of the tamping as is safe, say, to within 2 or 3 inches of charge, first having disconnected the wiring if electric battery is being used. Put in new primer with a small charge of high-grade explosive, retamp and fire, the small charge setting off charge below.

Loading.—Black blasting powder, after bore-hole has been cleaned, is placed in bottom and tamped with a wooden rammer. Fuse should reach some distance into charge and be carefully tamped in place; it should not be cut to required length until ready to fire. The entire charge should be well tamped, using dry sand, moist sand, fire clay, etc. Proper tamping very much increases the efficiency of the charge.

Dynamite charges are placed somewhat the same as powder, the first sticks being pressed into bottom of hole, then primer with fuse attached as near centre as possible. The remainder of charge is added, and finally tamped with clay or sand; ground brick also may be used. Many quarrymen contend that placing the primer on top or bottom gives just as good results. When firing with electric fuse care should be taken to properly connect lead wires to fuse, but never connect lead wires to battery until ready to fire. To prepare the primer when firing by safety fuse, cut end of fuse squarely across and place in cap, first making sure that fuse is perfectly dry. Be careful that, when pushing fuse in cap, no twisting occurs. The fuse should just touch the fulminate or the varnish protecting it. Then, with a pair of crimpers (and not with the teeth) fasten the cap on fuse. An opening is then made in dynamite stick with wooden plug and fuse placed in this opening. The cap should touch the dynamite on all sides since any space around it causes a cushion of air and lessens effect of detonation. The fuse is firmly tied in place. If the primer requires to be waterproof this may be done by using soap, tallow or wax. The string holding fuse in place should be of sufficient length to lower primer to bottom of hole. It should not be let down by fuse.

Mudcapping.—When blasting boulders by mudcapping them (also known by several other names), the charge of dynamite is packed closely against the surface on the top or side of the boulder, covered with mud and exploded. The charge should be placed on the spot which would be struck with a sledge if the boulder were small enough to be broken in that way. It should be packed in a solid mass by slitting the paper cartridge shells and massing them together, taking care not to spread them over the surface of the boulder any more than absolutely necessary. A blasting cap crimped into fuse should be placed in middle of the charge and the whole covered with six inches of damp clay or sand. This should be pressed firmly over the explosive, care being taken not to cover the outer end of fuse. If the boulder is deeply embedded in the ground, it is best, before blasting, to dig away or loosen some of the earth surrounding it.

If the boulder is cracked or seamy, the charge should be placed in some depression and covered with a quantity of clay or sand, as described before. This gives the explosive greater power.

A test made by Bureau of Mines at Washington to determine the effect of drilling and tamping holes in boulders, showed that a 1-inch hole 12 inches deep, with 1 ounce of explosive, had practically the same effect as 1 pound of unconfined explosive laid on the rock. The conclusion is that where time is not the principal object it is more efficient to drill. However, the cost of drilling should be against the cost of explosive. Three sticks of dynamite will practically cost the same as a 6-inch hole.

Quarrying.—The rock in quarries is generally covered with layer of earth which it is necessary to remove before blasting or separate from stone after blasting. This stripping will vary with the character of soil, depth of cut necessary, and the distance to which the material has to be moved. In estimating cost, this may be considered as ordinary earthwork.

Drilling.—This may be done either by hand, by steam, or compressed air. The two first named methods are generally used in Ontario. However, portable air compressors run by gasoline are used extensively in the States. One at Yonkers, N.Y., with an air cylinder 8 x 10 inches gave 96 cubic feet of free air per minute at 165 revolutions with 80 to 100 lbs. air pressure. A hoisting attachment for pulling rock from trenches, etc., was also connected on rear of machine. The entire outfit weighed about 8,000 lbs. Three hammer drills were used, each averaging 50 ft. per day. The cost per foot was approximately 14 cents.

Limestone drilling by hand costs anywhere from 35c. to 45c. per foot. That by steam from 20c. to 30c. a foot. In granite these prices are increased 25 to 30 per cent. In limestone three men will probably not drill more than 15 ft. per day. The steam drill will average about 45 ft. per day. In granite and trap rock these are correspondingly decreased 25 to 30 per cent. The estimates given above are for 1¼ inches steel.

Blasting.—There are few operations in quarrying where a workman can display a higher degree of skill and effect larger economies than in the proper placing of bore holes, and in the proper adjustment of his charges to the work to be done. Against the firing of one hole the following may be charged: labor, power (steam or air) where machine drills are used, wear and tear on plant, explosives, and a general loss distributed among many items in the operation of the quarry.

For the most part, efficient work in blasting is a matter of experience and good judgment. This cannot be taught in books, but there are some general rules which are fundamental, and in proportion as these are understood and appreciated the work of quarrying will be conducted with greater system and economy. Unsystematic efforts are always wasteful and costly, and system implies the recognition of some definite principles, according to which the work is laid out and prosecuted.

(1) The strength and quantity of the explosive should be properly proportioned to the cohesive strength or resistance of rock.

(2) The "burden," the shortest line that can be drawn from the charge in the bore-hole to the outer free face of rock, should bear a proper relation to the strength of the explosive and to the resistance of the rock.

(3) If the working face of rock is so blasted as to leave two or more free faces, instead of one for future blasts, the explosive required to overcome the resistance of the rock will be reduced.

(4) A seam or fissure may become important help, if hole is properly located, but the opposite may occur if charge is improperly placed.

(5) Breaking to regular benches and faces is most economical method, rock can be carefully observed for seams and fissures, admitting of a more intelligent placing of subsequent bore-holes.

(6) Simultaneous firing is more economical in most cases than firing singly or in series, for the reason that the adjacent charges assist each other reducing the amount of explosive required and the total length of holes to be drilled for any given volume of rock.

(7) Careful charging and tamping is absolutely essential. The more compact the charge the more efficient the results.

(8) The object in quarrying is to rupture rock, not to hurl it some distance. Hence only enough explosive should be used to accomplish this. Where rock is thrown great distance it is evident that the proper relation did not exist between the charge and the burden and that too large a charge was used for the length of the line of least resistance.

The amount of explosive required depends upon: (1) Kind of explosive used. (2) Depth of bore-hole. (3) Line of least resistance. (4) What stone is being used for.

From Gillette we have that the amount of 40% dynamite required per cubic yard of rock excavated varies with the depth of hole, decreasing as the depth of hole increases. In open cut work he uses the formula $P = 3/d$, in which P is equal to the pounds of dynamite required for cubic yard of rock and d depth in feet.

With 40% dynamite, at 15c. per lb., and drilling at 25c. per ft., we find upon summing up that the cost in cents per cubic yard of rock excavation, solid measure, using these rules, is as follows:—

When d (in ft.)	1	2	3	4	6	8	10	12
Cost of dynamite per cu. yd.	26	18	15	13	11	9	8	7
Cost of drilling per cu. yd...	100	66	34	29	15	11	9	9
Total cost per cu. yd.	126	84	59	42	26	20	17	16

The holes are generally spaced equal to their depth, but after 5 ft. in depth, Gillette uses the formula $S = 10/4 \sqrt{d}$; where S = spacing and d = depth.

OPERATION AND CARE OF MACHINERY. By W. Huber, B.A.Sc., assistant engineer, Ontario Office of Public Highways.

The two-fold effect of the introduction of various kinds of machinery into road building were stated to be (1) shortening the time required to make a finished road; (2) replacing a large percentage of labor of men and teams whose wages have been steadily increasing, thereby considerably reducing unit costs under favorable conditions. The author emphasized the importance of a careful selection of the machine, specially adapted for the work, of sufficient capacity and possessing the required degree of portability. The main requirements of the principal machinery used in road construction were given in brief as follows:—

Graders.—In the operation of these, mechanical traction is advised. The cost of operation of a steam or gasoline tractor is about equal to the wages of two teams. The amount of work is from 25 to 100 per cent. greater, depending upon its character. Skill, acquired only through experience, is necessary on the part of the operator, for efficient and scientific work.

The author advised against the practice of drawing sods to the centre of the road and leaving them there. They should be thrown out as they tend to prevent consolidation of the surface and to provide dirt and mud in wet weather.

Tractors.—The cost of team haulage varies, roughly, between 20c. and 30c. per yard-mile. Mechanical haulage averages approximately 10c. to 15c., while under favorable conditions large contracts have been accomplished in which the haulage was as low as 5c. per yard-mile. Economy demands that stoppages be reduced to a minimum. The tractor outfit is a paying proposition only when it is in continuous operation. This necessitates system in loading and unloading. Several devices were described by the author as suitable for saving time in these operations.

Steam vs. gas traction engines should be studied carefully. The former are more thoroughly understood and less complicated, although, with careful operation, the latter have been very successful and economical. It is essential that the operator knows his engine. In such case the gasoline tractor has a number of advantages, no time being required for starting, firing, etc.; also, the smoke and soot nuisance is eliminated.

Conditions producing greatest economy in the operation of mechanical tractors as opposed to team haulage are: long haul, fairly good roads and bridges and culverts in good repair. The mistake is often made of purchasing too heavy and too slow a machine, together with wagons of too large a capacity. A light tractor, capable of hauling about four 3-yard wagons at a speed of 3 miles per hour loaded, or four miles per hour empty, is more serviceable on work of this nature than one that will haul larger loads at a decreased speed.

Considerable advantage is attached to the use of reversible spreading wagons in 3-yard capacity, adaptable to either team or traction haulage.

The motor truck, while as yet confined largely to city work, will soon be found hauling materials for county roads. It is more easily manipulated than steam or gasoline tractors with their accompanying trains of wagons, and can travel at more than twice their speed. Two 5-ton trucks carrying, for example, 3½ or 4 cubic yards, could be made do the work of a 20-h.p. tractor hauling 14 to 16 yards per trip. A single truck can be made to do the work of about 4 teams.

Crushers.—The usual practice on county road work is to crush the stone and place it directly on the road. Economical practice requires that it be rolled and finished immediately after placing. In this way the crusher and roller are expected to work together. The amount of stone which a roller can consolidate into a finished road in one day is limited to from 60 to 100 cubic yards, depending on the quality of the stone, width and depth of metal, condition of subgrade, ability of engineer, etc. To supply more stone to the road each day than the roller can finish is to encourage partly finished and generally unscientific work. A crusher capable of turning out 100 cubic yards per day has been found the most satisfactory. Any surplus over the roller's capacity should be stored for future use, but the crusher should be worked to full capacity.

A crusher will not work efficiently when resting only on its wheels. The vibration will cause the axle bearings to wear, making the outfit draw heavier on the road. The size of all stone fed into the crusher should be such as to permit it to easily enter the jaws. Special attention must be given to lubrication. Automatic oiling devices, owing to the dust, have not been found reliable. Care should be taken to prevent dust from entering oil holes. For hot weather a heavier grade should be used than for cold.

Crusher bearings are subject to heavy wear. Wearing parts should be replaceable. Babbit bearings that can be changed in a few minutes are a decided advantage. Another factor influencing output is the condition of the jaw plates. They are usually of chilled cast iron and wear rapidly. In spite of a much higher cost, jaw plates of manganese steel are more economical, and owing to their durability, give a more uniform product.

The selection of screens is important. A portable bin with bucket elevator and rotary screen is indispensable. The latter should consist of two sections giving three sizes of stone. For limestone, the perforations should be 1 inch and 3 inches respectively. For granite or trap rock the 3-inch screen should be replaced by a 2½-inch screen.

For ordinary road work a 12-ton roller is sufficiently heavy. In many cases a 10-ton roller will do better work, depending upon the subgrade, which should not be disturbed. The relative merits of steam and gasoline apply to rollers as to traction engines. When steam is used a double cylinder engine is considered the the only satisfactory type. The cross compound engine supplied on some rollers of English manufacture possesses the same advantage together with additional ones, of being more economical of steam, and of permitting live steam to be turned into the low-pressure cylinder when extra power is required.

A roller with a low centre of gravity is more staple on highly crowned roads and on earth shoulders in slippery weather. There is a great difference in machines as to ease of manipulation, and this should be carefully considered when purchasing. The roller, being the most expensive item in the outfit, should be worked to its limit and the other parts of the organization should be regulated to suit its capacity. Its efficiency depends largely on the ability of the operator. The author showed clearly how poor firing and incompetent operation are detrimental to efficiency. The machine must be kept in first-class state of repair. Rear rolls should have a straight surface, and must be renewed, their life depending upon the iron, the hardness of the stone, etc. The necessity for renewing the rolls should be kept in mind when purchasing. Care of boiler and of the entire machine in all seasons is strongly emphasized.

CANADIAN MINING INSTITUTE.

At the recent meeting in Toronto of the Canadian Mining Institute the officers elected for the ensuing year were:— President, G. G. S. Lindsey, K.C.; vice-presidents, Thomas Cantley, New Glasgow, N.S., and Arthur A. Cole, Cobalt, Ont.; councillors, M. B. Baker, Kingston, Ont.; J. W. Bell, Montreal; R. W. Brock, Vancouver; T. Denny, Quebec; D. A. Dunlop, Toronto; M. B. R. Gordon, Cobalt; G. C. Mackenzie, Ottawa; D. H. McDougall, Sydney, N.S.; J. T. Stirling, Edmonton, and A. J. Young, Toronto.

Editorial

BEHAVIOR OF CONCRETE IN THE EDISON FIRE.

Probably the most thorough investigation that has ever been given the effect of intense fire upon reinforced concrete construction is that following the fire at the plant of Thomas A. Edison, Inc., at West Orange, N.J., on December 9th, 1914. At one time or other every type of fire-resistive material is called upon to vindicate, as in the above instance, the claims that are made for it, and, in the majority of cases, the lessons learned therefrom have a bearing upon the future practice in construction. In the case of the Edison fire there is much to be learned, particularly with respect to the expansion, as a unit, of a fire-infested building. Another point of great importance is the reported fusion of concrete, owing to the terrific heat developed. It is stated that the temperature in some of the buildings ranged between 2,000 and 2,500 degrees F., in certain stages of the conflagration. Owing to scarcity of water and to abundance of highly inflammable materials, this excessive temperature continued in places for several hours. Evidences of fused concrete were reported. This is a point, however, upon which the U.S. Bureau of Standards is devoting a great deal of exhaustive investigation at the present time, and its report upon the fusion will be awaited with universal interest.

The joint report of the National Board of Fire Underwriters and the National Fire Protection Association contains a very interesting section upon the effects of the fire on the concrete. It should be stated that the report attributes the severe loss to lack of adequate protective measures, in the matter of fire walls, fire resisting door and window construction, automatic sprinklers and water supply. A few items respecting the behavior of the concrete, however, are of particular interest. It states that the concrete panel walls withstood the fire excellently. There was no case of individual collapse, either, of a floor slab, beam, or girder, except in one basement where concrete and reinforcement were apparently melted away causing failure of three beams and a floor slab. All other failures are laid to the columns, and it is stated that their action was not as satisfactory as that of other concrete members. The column failures were largely due to longitudinal floor expansion. Another indication of failure is that the sudden and intense heat produced rapid expansion of the surface concrete, this causing severe internal stresses. Due to expansion of surface material, combined with stresses in different directions, at the junction of concrete members, a buckling effect of the concrete on the corners of the columns, together with shearing stresses in diagonal planes across these corners, resulted in a lengthwise splitting off of the corners, generally along the line of the reinforcing bars.

For the most part, corner columns remained in good condition, due probably to their being more heavily reinforced and possessing greater freedom of bending in the direction of the expansion of the building as a whole. Wall columns fared much worse, and many failed, due, it is thought, to the great difference in temperature between the inside and outside faces of the wall, and to their rigid position vertically which made them less able to resist the building expansion. Square columns in almost every case spalled at the corners, indicating a weakness of this shape of column when attacked by fire. Round columns suffered only minor damage.

While the report deals somewhat in detail with the behavior of the columns, it points out that more columns remained intact than were seriously injured. In summarizing, it observes that whether any other system of construction would have given better results under the same conditions, is problematical. "Reinforced concrete buildings can doubtless be built that would withstand such a fire satisfactorily, but no type of construction should be left to meet such an attack without the assistance of any of the standard fire-resistive measures which should be a part of every first-class building."

It is interesting to note the conclusions of the American Concrete Institute's report upon the disaster. It is in part as follows:—

"The fire fully demonstrated the advantages of monolithic structures. The fact that at five different places several of the wall columns were rendered useless and yet the upper portions of the building stood intact, is evidence of the superior merits of concrete in monolithic construction.

"Considering the extraordinary conditions surrounding this fire, the behavior of the concrete buildings was highly satisfactory and constitutes an excellent demonstration of the merits of concrete as a fire-resisting building material. It is not so surprising that the concrete buildings were damaged as that any material should have so satisfactorily withstood these unusual conditions.

"The end walls in the three upper floors of two buildings extended above the roofs of the adjoining buildings, which were completely destroyed; while this was in the hottest part of the fire, the walls were practically undamaged and are an admirable demonstration of the value of concrete walls as a fire barrier.

"The fused metal found in different parts of reinforced concrete buildings would seem to indicate that the fire reached an intensity of 1,000 degrees F. in all these buildings, and in many cases as high as 2,000 degrees F.

"In the greatest portion of these buildings the concrete remained firm and hard and intact after this severe heat treatment."

Relative to the net loss in the concrete buildings, Mr. Thomas A. Edison, in a letter, states:

"The report of our engineers shows that 87% of the reinforced concrete buildings, which were subjected to a very intense heat, are in good condition, and of the machinery which they contain about 85% can be used, with small repairs. Buildings of other materials, together with contents, were entirely destroyed."

It has been stated that the final revision of the Hudson Bay Railway route from Le Pas to Port Nelson will provide the most direct line in Canada for such a distance. It will be 424 miles long, 240 miles of which is completely graded and 54 miles partly graded. Trains will shortly be run to mileage 214. This year's work will likely complete grading operations to Port Nelson and the whole line will likely be ready for handling its share of the 1916 wheat crop.

SOME MODERN METHODS OF ARCH CALCULATION.

WITH many comprehensive textbooks upon this subject, it is manifestly impossible to deal fully with it in a short space. The following abstracts from a paper read by Ewart S. Andrews, B.Sc., at a recent meeting of the Concrete Institute (Great Britain) are given in the hope that, as the author stated in concluding his remarks, a study of them would enable those who have not had the opportunity of a complete study of the arch theory in its more general aspects to follow the fundamental relations upon which scientific calculations are based and that their interest will be sufficiently aroused to encourage them to study the subject more fully.

The arch is a form of structure which possesses great advantages from the standpoints of beauty and economy, and from the earliest times the arch has been used in all kinds of constructional work. In the present paper arches are not considered at all from the point of view of architectural styles or orders, consideration being restricted to the calculation of the stresses in them.

The arch presents points of considerable difficulty from this point of view, and the resulting formulæ are elaborate. Some may contend that the formulæ are too elaborate, and that "simple practical rules" are just as good. One answer to this contention is that, such simple rules would be welcomed if they really were as good. The difficulty is that, unless simple rules are applicable over a wide range, and have been fully tested by scientific experiment, there is considerable danger in their use.

There is, unfortunately, among some practical engineers a strong antipathy to complicated formulæ—an antipathy which is usually stronger in proportion as the formulæ are not understood—but closer acquaintance with such formulæ and some useful spade work in the form

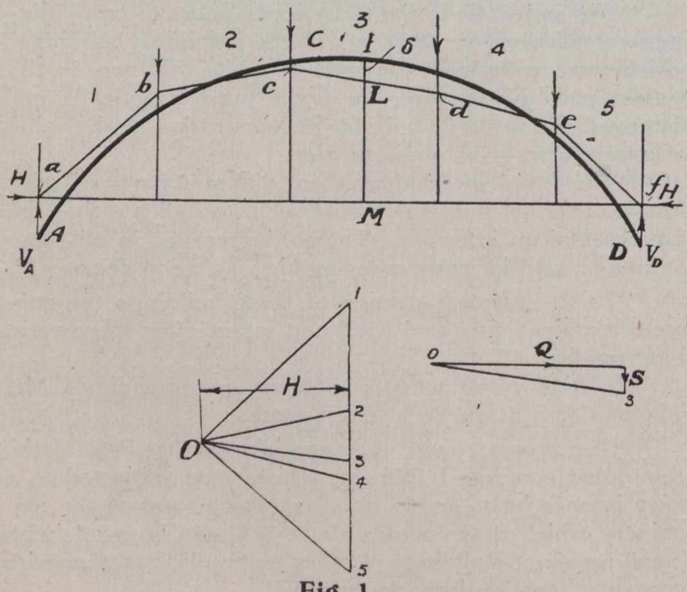


Fig. 1.

of the compilation of tables and diagrams usually helps to dispel much of the dread. The primary reason of the difficulty in the determination of the stresses in arches arises from the fact that in most cases the arch is what is called "a statically indeterminate structure," so that the forces acting upon the arch cannot be found by the ordinary laws of statics. Exactly similar difficulties arise in the case of stiffened suspension bridges, continuous beams, and slabs.

Determinations of Reactions in an Arch.—The stresses in an arch can be found as soon as we can find the magnitude and position of the reactions. These reactions R may be considered as compounded of vertical components V and horizontal thrusts H. If, as is usual, all the loads on the arch are vertical, the horizontal thrust H must be the same at each end. We can then draw the line of pressure, linear arch or equilibrium polygon (three alternative names for the same thing) for the given load system upon the arch.

Then, by Eddy's theorem that "the bending moment at any point of an arch is equal to the product of the hori-

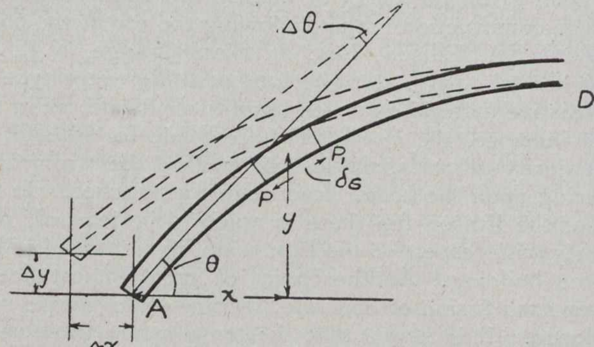


Fig. 2.

zontal thrust into the vertical intercept between the centre line of the arch and the line of pressure," we have—

Bending moment at any point P = H δ

If the line of pressure comes below the centre line of the arch, the upper surface or extrados of the arch will tend to become in tension, and if the line of pressure is above the centre line of the arch, the lower surface, or intrados, of the arch tends to become in tension.

Stresses in the Arch.—To obtain the stresses in the arch we first find the thrust or normal pressure Q, and the shearing force S, at the point by resolving the thrust O_3 at the point under consideration along and perpendicular to the centre line of the arch at the given point.

Then, if A is the area of the section, and M_c, M_t the compression and tension moduli, we have—

$$\text{Maximum tension stress} = t = \frac{H\,\delta}{M_t} - \frac{Q}{A} \quad . \ (1)$$

$$\text{Maximum compression stress} = c = \frac{H\,\delta}{M} - \frac{Q}{A} \quad . \ (2)$$

$$\text{Mean shear stress} = s = \frac{S}{A} \quad . \quad . \ (3)$$

There is therefore no difficulty whatever in the calculation of stresses in arches when once we know the horizontal thrust.

Cases in which the horizontal thrust can be found without the elastic theory of arches are as follows:—

(1) Parabolic arch uniformly loaded over whole span—

$$H = \frac{wl^2}{8v}$$

w = load per unit length, l = span of arch, v = rise of arch

(2) Parabolic arch with uniform load extending from an abutment to the centre—

$$H = \frac{wl^2}{16v}$$

(3) Arch of any shape provided with hinges.—In this case the line of pressure can be drawn by the well-known graphical construction for making a link-polygon pass through any three given points; then the polar distance of the vector diagram gives the horizontal thrust desired.

Elastic Theory of Arches.—This theory, to which the author's principal attention was given, is based upon a consideration of the deformations which an arch-rib obeying the ordinary laws of bending will receive. From these deformations we are able to calculate the horizontal thrusts.

Let δs represent a very short length between two points of an arch rib, one end of which is considered as fixed relatively to the other end, and suppose that the bending moment along this very short length is B.

If E is the elastic (Young's modulus), and I the moment of inertia of the rib, it can be shown by the theory of beams that—

Δy = vertical displacement of D due to bending

$$= \sum_{A}^{D} \frac{B\, x\, \delta s}{E I} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (1)$$

Δx = horizontal displacement of D due to bending

$$= \sum_{A}^{D} \frac{B\, y\, \delta s}{E I} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (2)$$

$\Delta \theta$ = angular change of tangent to rib at D

$$= \sum^{D} \frac{B\, \delta s}{E I} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \qquad (3)$$

These are the general formulæ upon which all the special formulæ are based, and it should be noted in passing that they are the deformations due to bending only, and do not include those due to direct thrust.

These general formulæ were then applied to the most common special cases of rigid arches, viz.,:—

(1) Arches with two hinges or pin joints—*i.e.*, two-pinned arches.

(2) Arches without hinges—*i.e.*, fixed arches.

RUBBER A BY-PRODUCT FROM STEEL.

At a recent meeting of the Iron and Steel Institute in London, the president read a paper on by-products in steel manufacture. He discussed the utilization of blast furnace gases for operating gas engines and for illumination and heating, and the later developments in making nitric acid from these gases, and also the manufacture of bricks and cement from slags. A new idea was presented looking toward the synthetic production of india rubber from coke oven gases. The president's words were: "It was being sought to obtain from it the hydrocarbons, the derivatives of which were found in india rubber, and experiments that had been made permitted the foreshadowing of the manufacture of artificial rubber."

THE PREVENTION OF CORROSION.

The treatment of iron or steel to prevent rusting and to remove grease and oil before painting them is the subject of a United States Patent No. 1119781. The cleaning mixture consists of orthophosphoric acid and denatured alcohol, the proportion, 1 volume (87 per cent. solution) of the former to 2 volumes of the latter (ethyl alcohol), being suitable for ordinary sheet steel work. The addition of quarter volume of carbon tetrachloride destroys the inflammable nature of the mixture. The result of the action of the acid in this alcohol is the formation of an alcoholic phosphate. This is painted over the metal treated, and after a few minutes the metal is wiped. It is stated that the cleaner dissolves rust and changes grease, etc., into harmless substances, which are wiped off the metal.

Coast to Coast

Edmonton, Alta.—Mr. A. J. Latornell, City Engineer, is making a test of the bituminous sands of the Mackenzie River. The Department of Mines, Ottawa, is having 60 tons transported from Fort McMurray, a distance of 252 miles, by teams, to the C.N.R. line at Athabasca Landing.

Hamilton, Ont.—On January 12th, the Steel Company of Canada placed an order with the Hamilton Bridge Works Company for the erection of a building 150 ft. x 90 ft., and 50 ft. in height. The building was erected in three weeks, about 100 tons of steel being used. It has been equipped for the forging of shrapnel shells.

Peterborough, Ont.—The city will pay $154,616 to the Peterborough Light and Power Company for its entire distributing system, according to an arbitration award made on March 5th. Peterborough ratepayers adopted the principle of hydro-electric power four years ago. One of the arbitrators in the above instance was R. A. Ross, of Montreal.

Hamilton, Ont.—Residents of the west end of the city and T.H. & B. authorities have arranged for a meeting this week with Mr. Geo. A. Mountain, of the Dominion Railway Board, to discuss plans for changing the grade of the T.H. & B. tracks. The plans submitted by the company provide for the closing of Charles, MacNab and Hughson Streets.

Ottawa, Ont.—According to the annual report of Hon. Frank Cochrane, Minister of Railways and Canals, the total expenditure on the Transcontinental Railway to the end of the last fiscal year was $142,968,000. Track laying was completed on November 17th, 1913. There are 1,803 miles on the main line, 20 miles of double track and 423 miles of sidings. Steel bridges were 97.2 per cent. completed in March last.

Quebec, Que.—The Dorchester bridge which should have been completed and opened for traffic in March, 1914, was the subject of discussion by the city council recently, and the contractor was requested to have it wholly completed by the 15th of May next. Several bridge engineers stated that it was quite safe for traffic at the present time, when called in to report in connection with a request from the contractor that the bridge be accepted as it now stands. It was decided, however, to require the contractor to finish the structure completely.

Calgary, Alta.—A case which involves over $100,000 is pending at the Supreme Court in Calgary. Mr. Z. Malhoit, a well-known engineer of the city, claims that he prepared plans, specifications and compiled other valuable information relative to a water power development on the Bow River, east of Calgary. For these he was to receive $20,000 from each of five defendants, on January 1st, 1914. He is suing for this amount, and additional $1,000 as compensation for alleged breach of contract.

Saskatoon, Sask.—G. D. Archibald, City Engineer, reports that the expenditures of his department in 1914 amounted to $478,791, less than half the amount for the previous year. The work done since 1909 is also given, and includes the following:—45.32 miles of water mains, 42.40 miles of sanitary sewers, 2,738 water connections, 2,734 sewer connections, 465 hydrants, 52 miles of storm sewers, 56 miles of graded streets, 5.76 miles of paved streets, 53.82 miles of concrete walks, 9.7 miles of plank walks, 3.18 miles of intercepting sewer (not in use). During the year 1.49 miles of water mains were laid and .9 mile of sewer was laid.

CANADIAN SOCIETY OF CIVIL ENGINEERS.

Regular Meeting, March 4th, 1915.

The regular monthly meeting of the Canadian Society of Civil Engineers was held in Montreal on the evening of the 4th instant. A paper of unusual merit by Mr. Chas. A. Lee, A.M.Can.Soc.C.E., of Vancouver, describing the Jordan River Power Development, was presented. In the absence of the author, the paper was read by Mr. de Gaspé Beaubien, A.M.Can.Soc.C.E., of Montreal. At the conclusion a number of lantern slides were shown. Mr. W. F. Tye, Past President of the Society, was Chairman.

The many interesting and unique points brought out in the paper were of so much importance that it was decided to defer the discussion of the paper until a subsequent meeting.

Mr. Lee described graphically the conditions in the primeval forests of the Island of Vancouver, the difficulties of transportation, and the tremendous rainfall and percentage of run-off. He also gave a full description of the second highest reinforced concrete skeleton dam that has ever been built, and concluded with a full account of the power house and equipment. The head under which the plant operates is one thousand one hundred and fifty feet.

At the conclusion of the address a hearty vote of thanks was tendered to Mr. Beaubien for the lucid manner in which he read the paper, and it was moved by Mr. J. A. Jamieson, and seconded by Mr. Frederick B. Brown, that the thanks of the Society be tendered to Mr. Chas. A. Lee for his exceedingly interesting and valuable contribution to the literature of the Society.

HALF A THOUSAND CANADIAN ENGINEERS FOR BELGIUM AND FRANCE.

Arrangements are being made at Ottawa for the organization of a special corps of 500 engineers to go to the front for the purpose of rebuilding bridges, and readjusting railway lines throughout the war zone. Mr. C. W. P. Ramsey, engineer in charge of construction for the Canadian Pacific Railway, is at present in charge of the organization work, and it is probable that he will be in command of the Corps. The great importance of rapid lines of communication in modern warfare necessitates competent repair and construction of existing lines as well as speedy replacement of destroyed bridges and works of a similar nature. The corps will consist of railway and bridge engineers, and mechanics experienced in work of this nature. While Mr. Ramsey is the C.P.R. man, the corps itself will consist largely of men from the three large Canadian systems.

PERSONAL.

WM. MAUND, Travelling Auditor for the T. & N. O. Railway, has been appointed to succeed the late A. J. McGee as Secretary and Treasurer of the T. & N. O. Commission.

G. G. S. LINDSEY, K.C., of Toronto, was re-elected president of the Canadian Mining Institute at its convention last week. Other officials are announced elsewhere in this issue.

W. H. HAZLETT, for 15 years with the British Columbia Electric Railway Company, has joined the firm of A. G. Langley & Company, Limited, engineers and contractors, Vancouver, B.C.

REG. H. THOMSON, engineer at Strathcona Park, B.C., for the British Columbia Government, is reported to have sent in his resignation. It is understood that he will engage in business in Seattle.

HENRY W. DURHAM, M.Am.Soc.C.E., former chief engineer of highways, Borough of Manhattan, New York City, on February 26th, delivered an illustrated lecture on "Relation of Tests of Physical Properties and the Wearing Qualities of Paving Materials," before the graduate students in highway engineering at Columbia University.

E. L. COUSINS, B.A.Sc., Chief Engineer of the Toronto Board of Harbor Commissioners, has been granted leave of absence to undertake the preparation of plans for a system of radials and radial entrances for the city. It is estimated that this work will engage Mr. Cousins' services for 8 or 9 months. During this time Mr. Jas. Wainwright, Assistant Harbor Engineer, will have charge of the harbor improvements.

JNO. W. ASTLEY, C.E., who for the past seven years has been engineer of construction for Winnipeg, is severing his connection with the city in that capacity. Winnipeg, in view of the necessity of curtailing practically all work except absolutely necessary expenditure, has decided to go back to the old order of things and place all such work under the City Engineer's department. During his lengthy tenure of office with the city, Mr. Astley had a very large experience in handling almost every class of construction work. It will be remembered that he is a strong advocate of day labor for certain classes of city works, especially local improvements.

OBITUARY.

The death occurred in Toronto last week of Mr. A. J. McGee, secretary-treasurer of the Timiskaming and Northern Ontario Railway Commission. Mr. McGee had served the province in this capacity since 1905, having previously been associated with the Canada Atlantic Railway. His death is most deeply felt in Northern Ontario, in the development of which he took a very active interest.

The death occurred on February 17th, of Mr. Wm. F. Gurley, president of the firm of W. & L. E. Gurley, Troy, N.Y., with whose products Canadian engineers and surveyors are familiar. Mr. Gurley was a noted educationalist and philanthropist, a staunch advocate of thoroughness in commercial and financial development, and an active participant in civic and state affairs. He was president of the Troy and Bennington Railroad, treasurer of the Taylor Truck Company, and a director of many commercial, banking and educational institutions.

COMING MEETINGS.

AMERICAN RAILWAY ENGINEERING ASSOCIATION.—Annual meeting to be held in Chicago, March 16th to 18th, 1915. Secretary, E. H. Fritch, 900 South Michigan Avenue, Chicago.

CANADIAN AND INTERNATIONAL GOOD ROADS CONGRESS.—Second Annual Convention, Toronto, March 22 to 26, 1915. Secretary, Geo. A. McNamee, Dominion Good Roads Association, Montreal.

TORONTO ELECTRICAL SHOW.—The second annual exhibition, to be held in the Arena, Toronto, April 12th to 17th. Secretary, Mr. E. M. Wilcox, 62 Temperance Street, Toronto.

AMERICAN WATERWORKS ASSOCIATION.—The 35th annual convention, to be held in Cincinnati, Ohio, May 10th to 14th, 1915. Secretary, J. M. Diven, 47 State Street, Troy, N.Y.