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OF GENERAL INTEREST

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CONTINUOUS FRAME ANALYSIS OF FLAT SLABS

BY DEAN PEABODY, JR., MEMBER*

(Reprint of Article in July 1939 B.S.C.E. Journal with increase of illustrative problems.)

THE American Codes for reinforced concrete design have contented themselves in the past with recommending for flat slabs that shear forces and bending moments be obtained by methods of analysis that allow for the continuity of the structure. Definite moment coefficients have only been suggested for the case where the successive spans are approximately equal ("Approximately" being defined to allow a 20 per cent variation in length over the shorter span).

It would seem that the time has come when more precise methods of moment determination are justified for structures with floor systems whose successive spans are markedly unequal. This is particularly needed for flat slab floors and for 2-way floors.

The mathematicians have worked on solutions for a long time and recently the engineer-mathematicians have tied together theory and tests with a seasoning of practical experience. The German specifications for flat slabs are based on the elastic web analysis of Dr. H. Marcus, which is briefly summarized by Professor J. A. Wise in the 1928 A.C.I. Proceedings. The American Concrete Institute and Joint Committee moment coefficients are based on the analysis of Dr. Westergaard with a survey of tests by Professor Slater (1921 A.C.I. Proceedings). Dr. Westergaard submitted his results for use in the form of floor plans with appropriate moment coefficients for 2-way and flat slabs with equal spans (see also 1926 A.C.I. Proceedings).

It is, however, desirable to have a general method of analysis

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that can be used by the designer to obtain for himself reasonable coefficients for unusual span ratios. The 1932 German specifications state, "The slab is resolved along the rows of the columns, transverse and longitudinal, with two series of continuous beams, framed into each other with the same degrees of restraint at the columns, as prevail in the slab proper", and the procedure of analysis is left to the designer aided only by a paragraph of suggested approximations.

Similarly, the 1937 Pacific Coast Building Official Conference Code states for design of flat slabs by Method B, "Flat slabs . . . may be designed by treating such slabs, together with their supporting columns and walls, as elastic frames . . .", followed by a page of general assumptions. The Code adds "In lieu of the elastic frame analysis . . . any exact method of analysis may be used, subject to approval. . . ."

Flat slabs have been designed in Germany since 1932 and in California since 1937 according to these general statements. The facts that the buildings have behaved satisfactorily and that the Joint Committee coefficients for equal spans can be obtained by a general method of elastic frame analysis are justification for its use.

Elastic Frame Analysis

Elastic frame analysis involves the use of slope and deflection equations in some one of the variations in general use. The basic equation may be written in the form.

$$M_{ab} = 2EK\ (2\theta_a + \theta_b - 3R) \pm F.E.M. \qquad (1)$$

where M_{ab} = the bending moment at the end A of a length AB

E = modulus of elasticity

K = ratio $\frac{I}{l}$

$\theta_a = \theta_b$ = change of slope at sections A and B

R = ratio $\frac{d}{l}$, where d = deflection of B relative to A

$F.E.M.$ = the bending moment at A if sections A and B are fixed.

This basic equation is often employed by the variation known as Moment Distribution. This is the method that will be used in this discussion. For reinforced concrete design:

I, the moment of inertia, is taken as the full sectional area with no allowance for the steel.

E is regarded as a constant for elastic loading, which is true for the richer mixes. When the effect of shrinkage, temperature changes, and plastic flow are included, the correct value of E is somewhat problematical and it can best be regarded as an empirical constant based on tests that allow for all these effects. In such cases the symbol E is really the modulus of resistance $R = \frac{n}{e+c}$ where:

n = normal stress intensity
e = elastic strain, and c = plastic flow, shrinkage, or temperature strains.

Simplifying Assumptions—Flat Slabs or 2-Way Slabs

1. All joints of columns and slabs are rigid.
2. There is no change in length of a member due to direct stress.
3. No deflection of a member due to internal shear stress.
4. Span of slabs shall be taken as center to center of columns.
5. Height of columns as center to center of floor slabs.
6. Width of slab equals width of floor bay.
7. Moment of inertia of any section shall be taken as the homogeneous gross section neglecting reinforcement.
8. The structure may be considered divided into a number of bents, each consisting of a row of columns and strips of supported floor slab-systems, each strip bounded laterally by the center line of the panel either side of the row of columns. The bents shall be taken longitudinally and transversely of the building.
9. Each such bent may be analyzed in its entirety, or each floor thereof and the roof may be analyzed separately into its adjacent columns above and below, the columns being assumed fixed at their remote ends. Where slabs are thus analyzed separately in bents more than four panels long, it may be assumed in determining the bending at a given support that the slab is fixed at any support two panels distant therefrom, beyond which the slab continues.
10. Supports of columns shall be assumed free from settlement or movements, unless the amount thereof is determined.

This paper discusses the practical application of continuous frame analysis to flat slabs with unequal spans. Much of the nomenclature and the results are taken from an extended analysis made by Mr. R. L. Bertin, chairman of the A.C.I. sub-committee on flat

slabs, while studying the form of a revision of the present chapter 10 of the A.C.I. Code.

Flat Slabs

$\frac{M}{EI}$ Diagrams

An essential part of the use of slope-deflection method is the $\frac{M}{EI}$ diagram. In Figure 1, is shown a characteristic flat slab floor system

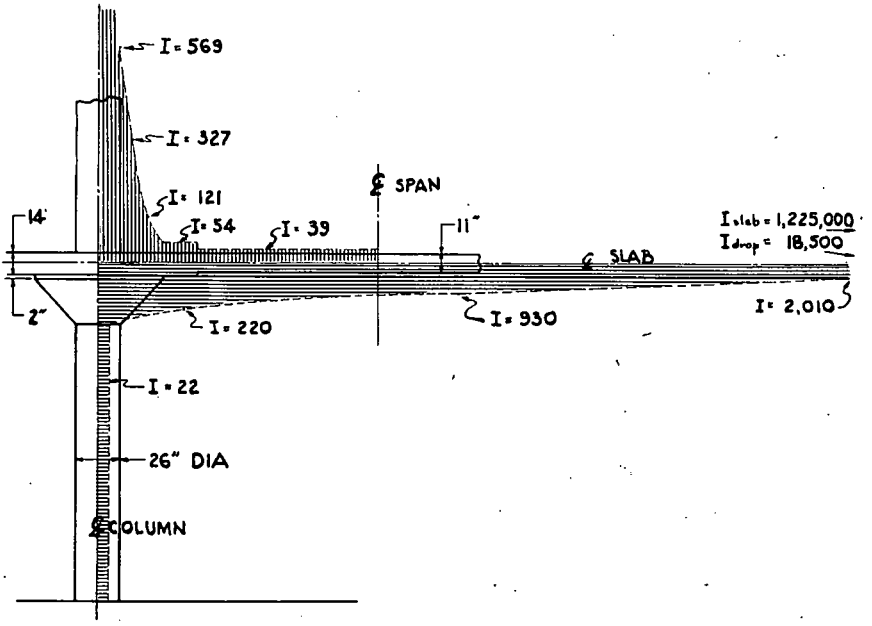
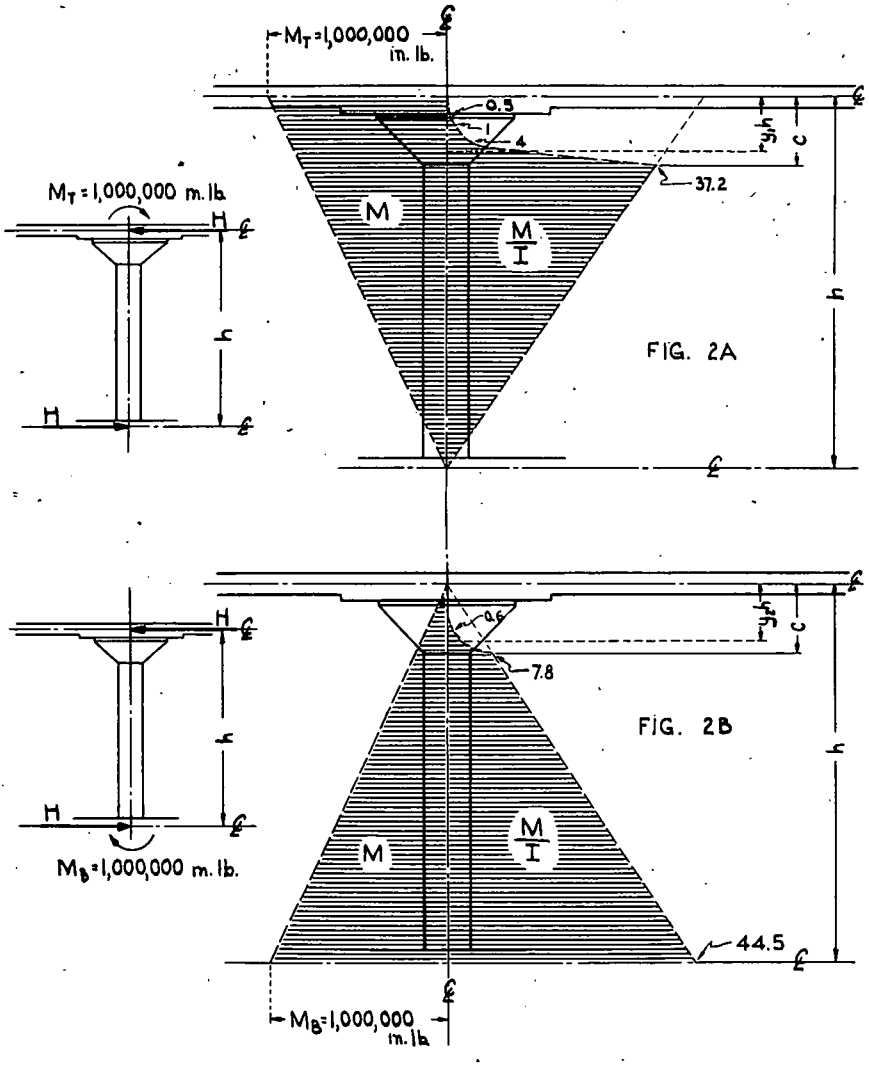


FIG. 1 - FLAT SLAB VARIATION OF MOMENT OF INERTIA IN COLUMN AND SLAB. COLUMN CAPITAL = 80 INCHES IN DIAMETER; DROPPED PANEL = 122 INCHES SQUARE

with the shaded area representing the moment of inertia, I, of the column and the slab, computed from the gross section with the reinforcement disregarded. These moments of inertia have been computed for the columns taking the sections perpendicular to the column center line from the junction with the slab center line to the junction with the slab center line above. Similarly the slab moment of inertia has been figured up to the center line of the column using a width of

slab equal to the column spacing laterally. In both cases the moment of inertia increases tremendously as the column-slab junction is approached.

In Figure 2 the column is assumed to be supported at the ends and subjected to a couple M applied at the capital (Figure 2A) or at



FIGS. 2A & 2B – $\frac{M}{I}$ DIAGRAMS FOR COLUMNS

the base (Figure 2B). In each case the moment varies uniformly to zero at the other end. There also is plotted in Figure 2 the values of $\frac{M}{I}$. These vary uniformly as long as the sections are taken in the column but decrease rapidly in the capital. The $\frac{M}{I}$ diagram can be approximated by assuming it to be uniformly varying to a section yh from the upper end. This distance yh should be of such a value that the moment of the approximate $\frac{M}{I}$ diagram about the slab center line equals the moment of the actual $\frac{M}{I}$ diagram, so that the same end slope θ is obtained. This will locate a section near the bottom of the capital but for practical design it is sufficiently accurate to assume yh is the distance from the bottom of the drop panel to the bottom of the capital. In this case the moment of inertia I is the constant value of the column proper.

Slope at Ends of Column—Simply Supported

If such a simplified $\frac{M}{EI}$ diagram is assumed, the slopes at the column ends can be computed. In Figure 3A the conjugate beam is shown with a couple applied at the upper end. The supporting forces for the conjugate beam are the slopes at top (θ_T) and bottom (θ_B). Similarly, in Figure 3B is shown the conjugate beam for a couple applied at the bottom end of the column.

Couple M_T at Upper End of Column

At a distance yh below the upper support the moment

$$M_y = M_T\,(1 - y) \qquad \text{(Fig. 2A)}$$

$$\text{Area of } \frac{M}{EI} \text{ diagram (Fig. 3A)} = \frac{M_y h}{2EI}(1-y) = \frac{M_T h}{2EI}(1-y)^2$$

$$\therefore\ \theta_T = \frac{M_T h}{2EI}(1-y)^2\left[(1-y)\frac{2h}{3} \times \frac{1}{h}\right]$$

$$= \frac{M_T h}{3EI}(1-y)^3 = \frac{M_T h}{3EI} f_1 \qquad (1)$$

$$\theta_B = \frac{M_T h}{6EI}(1-3y^2+2y^3) = \frac{M_T h}{6EI} f_2 \qquad (2)$$

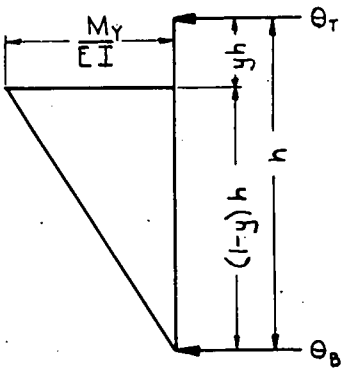


FIG. 3A

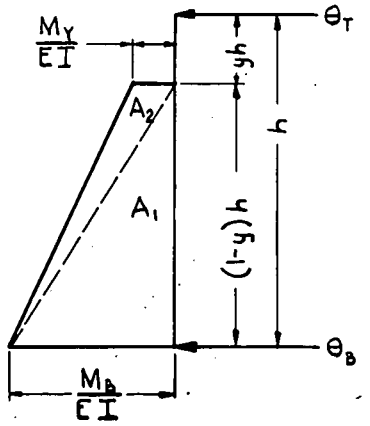


FIG. 3B

Couple M_B at Lower End of Column

At a distance yh below the upper support the moment

$$M_y = M_B y \qquad \text{(Fig. 2B)}$$

Area of $\frac{M}{EI}$ diagram (Fig. 3B) $= A_1 + A_2$

$$\therefore\ \theta_T = \frac{M_B h}{2EI}(1-y)\left[(1-y)\frac{h}{3}\times\frac{1}{h}\right]$$

$$+\frac{M_B h}{2EI}(1-y)\left[(1-y)\frac{2h}{3}\times\frac{1}{h}\right]$$

$$\theta_T = \frac{M_B h}{6EI}(1-3y^2+2y^3) = \frac{M_B h}{6EI} f_2 \qquad (3)$$

$$\theta_B = \frac{M_B h}{3EI}(1-y^3) = \frac{M_B h}{3EI} f_3 \qquad (4)$$

Couple at One End of Member of Constant Moment of Inertia

$$\theta_{near} = \frac{Mh}{2EI}\left(\frac{2h}{3}\times\frac{1}{h}\right) = \frac{Mh}{3EI} \qquad \text{(Fig. 4)} \qquad (5)$$

$$\theta_{far} = \frac{Mh}{2EI}\left(\frac{h}{3}\times\frac{1}{h}\right) = \frac{Mh}{6EI} \qquad (6)$$

Equations (1) and (4) differ from equation (5) by term f_1 or f_3. Equations (2) and (3) differ from equation (6) by term f_2.

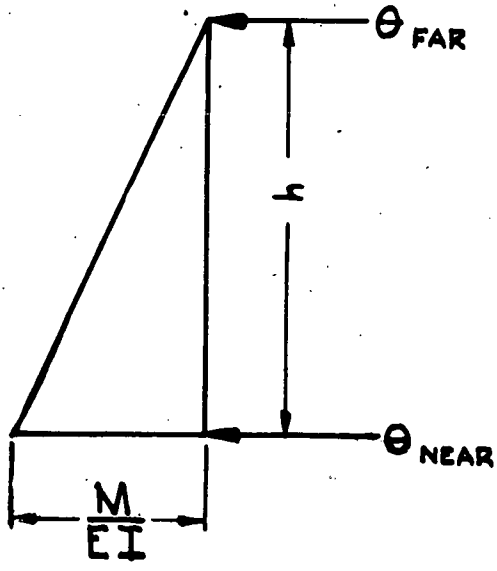


FIG. 4

Figure 5 shows variation of the (f) terms with values of (y) varying from $y = 0$, when all (f) terms equal unity [see equations (5) and (6)], to $y = 0.2$.

The dash lines give approximate values of the (f) terms according to the equations:

$$f_1 = 1—2.5y \tag{7}$$
$$f_2 = 1—0.5y \tag{8}$$
$$f_3 = 1.0 \tag{9}$$

These values will be used in the succeeding discussion.

Moments at Ends of Column—Far End Fixed

If a couple M_T (Fig. 2A) is applied at the top of the column and the bottom is fixed, a couple M'_B at the bottom causes the slope θ_B to become zero. Equations (2) and (4) must sum up to zero, and:

$$M'_B = \frac{M_T}{2} \frac{f_2}{f_3} = \frac{M_T}{2} (1—0.5y) \tag{10}$$

If a couple M_B (Fig. 2B) is applied at the bottom and the top

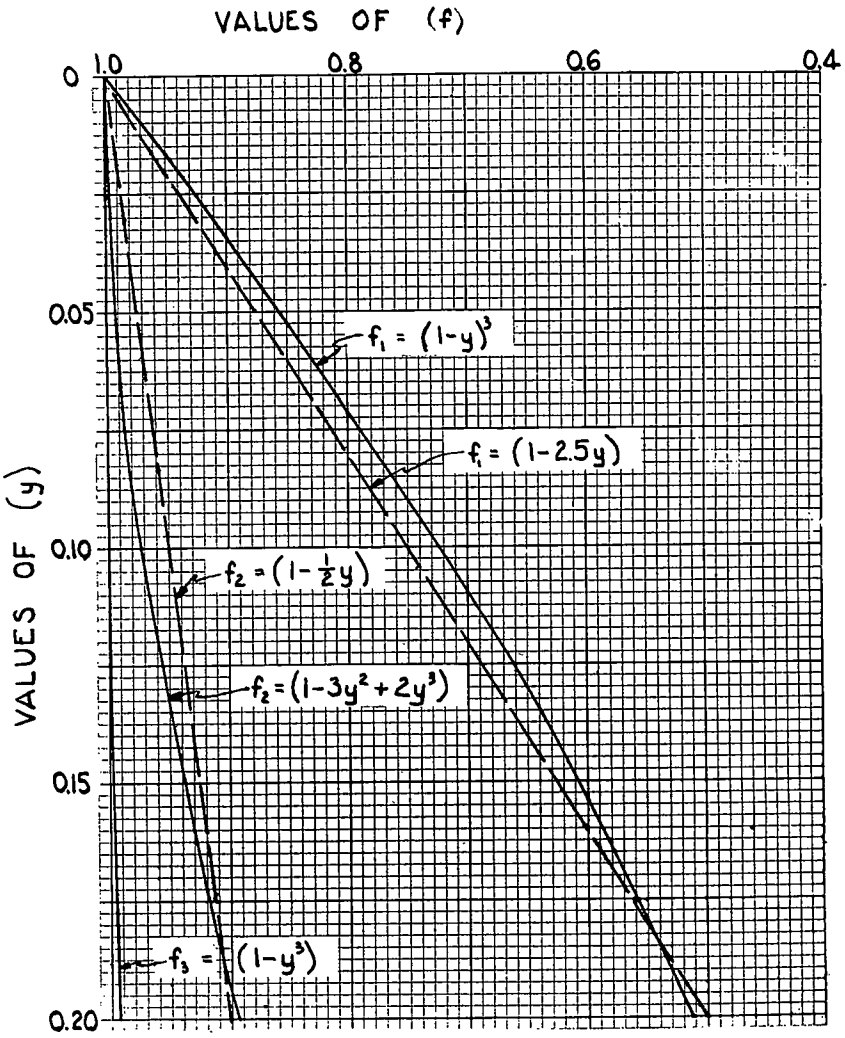


FIG. 5 - COLUMN COEFFICIENTS

of the column is fixed, a couple M'_T at the top causes the slope θ_T to become zero. Equations (1) and (3) must sum to zero, and:

$$M'_T = \frac{M_B}{2} \frac{f_2}{f_1} = \frac{M_B}{2} \frac{(1-0.5y)}{(1-2.5y)} \tag{11}$$

If the member had a constant moment of inertia (Fig. 4), the fixed end moment, M'_{far} would be:

$$M'_{far} = \frac{M}{2} \tag{12}$$

Slab— $\frac{M}{EI}$ Diagram

The variation of the moment of inertia of the slab is shown in Figure 1. Assuming the slab to be supported at the center line of the columns and that a couple $M = 1{,}000{,}000$ in. lb. is applied at the left support, the $\frac{M}{I'}$ values have been plotted in Figure 6. I' is the actual moment of inertia at each section. This irregular area may be approximated by a uniformly varying diagram of $\frac{M}{I}$ (shown hatched in Figure 6) starting with a value of $\frac{M_x}{I}$ at a distance xl from the left support and ending with a value of $\frac{M'_x}{I}$ at the same distance xl from the right support. The moment of inertia I is that of the slab in the central portion of the span.

The slopes at θ_l at the left and θ_r at the right end can be determined by using the original irregular $\frac{M}{I}$ diagram as the loading of a conjugate beam. The distances xl should be of such value that the substitute $\frac{M}{I}$ diagram will give the same end slopes θ_l and θ_r. Mr. Bertin recommends that:

$$xl = b - (b-a) \frac{I}{I_1} \tag{13}$$

where: a = distance from column center-line to edge of capital
b = distance from column center-line to edge of dropped panel.

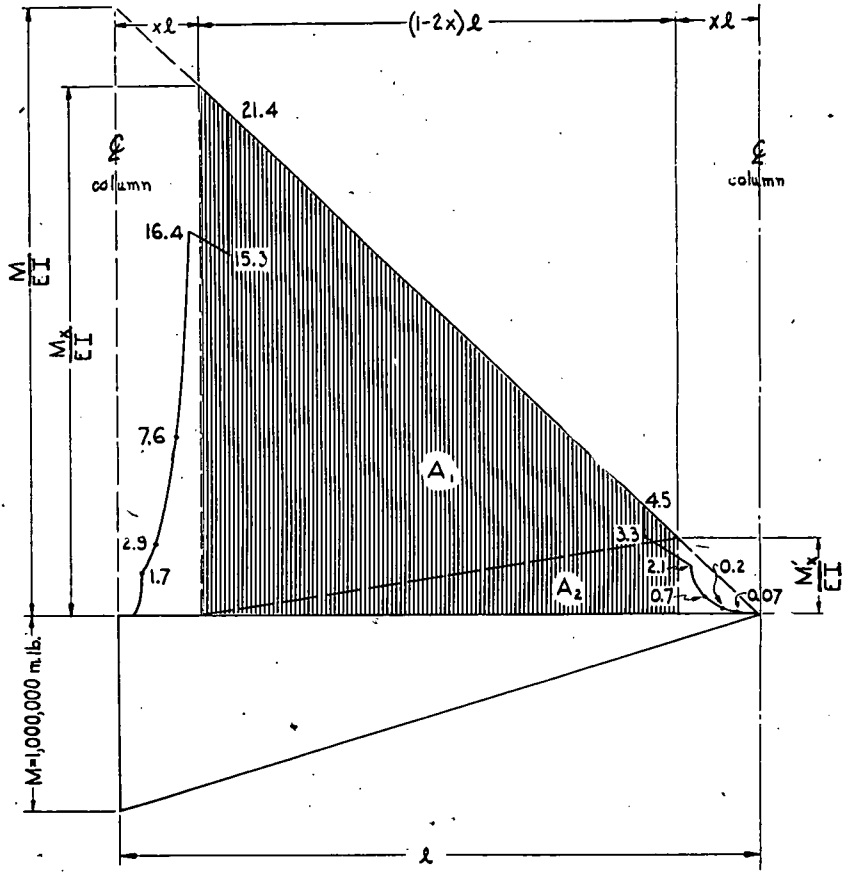


FIG. 6 - $\frac{M}{I}$ DIAGRAMS FOR THE SLAB

I_1 = moment of inertia of sections through the dropped panel.

I = moment of inertia of sections through slab.

Slab—Couple at One End

If the portion of the $\frac{M}{I}$ diagram within the distance xl of the slab supports be neglected, as in the approximation above, the slopes

at the end of a simply supported beam can be determined for a couple applied at one end. In Figure 6 the hatched area will be the $\frac{M}{I}$ diagram for a conjugate beam with couple M applied at the left end. Dividing this hatched area into two parts A_1 and A_2, the slope θ_{near} at the left end can be found.

at xl from left end: $$M_{xl} = \frac{M}{EI} \frac{l-xl}{l} = \frac{M}{EI}(1-x) \tag{14}$$

$$A_1 = \frac{M}{EI}(1-x)\left[\frac{(1-2x)l}{2}\right] \frac{Ml}{2EI}(1-x)(1-2x) \tag{15}$$

at xl from right end: $$M_{(1-x)} = \frac{M}{EI} \cdot \frac{xl}{l} = \frac{M}{EI}x \tag{16}$$

$$A_2 = \frac{Mx}{EI}\left[\frac{(1-2x)l}{2}\right] = \frac{Ml}{2EI}x(1-2x) \tag{17}$$

Then: $$\theta_{near}l = \frac{Ml}{2EI}(1-x)(1-2x)\left[xl + \frac{2}{3}(1-2x)l\right]$$

$$+ \frac{Ml}{2EI}x(1-2x)\left[xl + \frac{(1-2x)l}{3}\right]$$

$$\theta_{near} = \frac{Ml}{3EI}(1 - 3x + 3x^2 - 2x^3) = \frac{Ml}{3EI}f_4 \tag{18}$$

Similarly: $$\theta_{far}l = \frac{Ml}{2EI}(1-x)(1-2x)\left[xl + \frac{(1-2x)l}{3}\right]$$

$$+ \frac{Ml}{2EI}x(1-2x)\left[xl + \frac{2}{3}(1-2x)l\right]$$

$$\theta_{far} = \frac{Ml}{6EI}(1 - 6x^2 + 4x^3) = \frac{Ml}{6EI}f_5 \tag{19}$$

Slab with Constant Moment of Inertia

For the beam with constant moment of inertia throughout a span of l

$$\theta_{near} = \frac{Ml}{3EI} \qquad \text{(equation 5)}$$

$$\theta_{far} = \frac{Ml}{6EI} \qquad \text{(equation 6)}$$

Slab—Ends Fixed

If the far end is fixed, a couple M' at that end is acting to cause the slope θ_{far} to become zero. By equation (18), M' acting alone would cause a slope of

$$\theta_{far} = \frac{M'l}{3EI} f_4$$

The original couple M produces a slope of

$$\theta_{far} = \frac{Ml}{6EI} f_5$$

The sum of the two slopes must be zero, so:

$$M' = \frac{M}{2} \frac{f_5}{f_4} \tag{20}$$

Slab—Loaded with Uniformly Distributed Load

If the portion of the $\frac{M}{EI}$ diagram within the distance xl of the slab supports be neglected, the slopes at the ends of a simple supported slab can be determined for a uniformly distributed load on the slab. In Figure 7 is shown the parabola which is the bending moment diagram. The hatched portion is to be used in computations. Since the $\frac{M}{EI}$ diagram is symmetrical the two end slopes are equal.

$$M_{xl} = \frac{wl}{2}(xl) - \frac{w}{2}(xl)^2 = \frac{wl^2}{2} x(1-x)$$

$$\text{Area } A_1 = \frac{M_{xl}}{EI}(1-2x)l = \frac{wl^3}{2EI} x(1-x)(1-2x) \tag{21}$$

$$\text{Area } A_2 = \left(\frac{wl^2}{8} - M_{xl}\right)\left[(1-2x)l\right]\frac{2}{3EI}$$

$$= \frac{wl^2}{8EI}(1-4x+4x^2)(1-2x)\frac{2l}{3}$$

$$= \frac{wl^3}{12EI}(1-2x)^3 \tag{22}$$

$$\text{End slope } \theta = \frac{A_1 + A_2}{2}$$

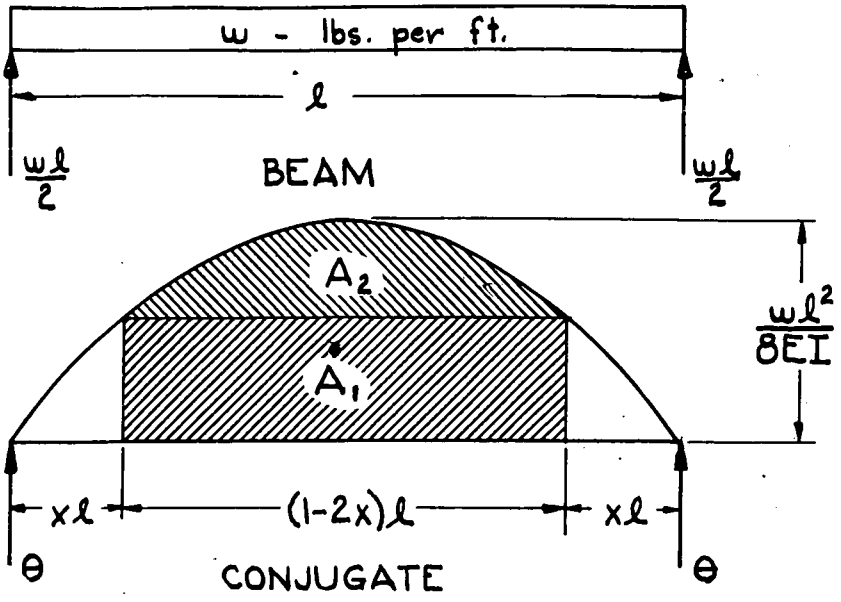


FIG. 7

$$= \frac{1}{2EL}\left[\frac{wl^3}{2}x(1-x)\,(1-2x) + \frac{wl^3}{12}(1-2x)^3\right]$$

$$\theta = \frac{wl^3}{24EI}(1-6x^2+4x)^3 = \frac{wl^3}{24EI}f_5 \qquad (23)$$

Slab Fixed at Both Ends

If the slab is fixed at both ends and loaded with a uniformly distributed moment, the fixed end moment must produce zero slope at the ends. By equations 18, 19, and 23 this moment M' equals:

$$M' = \frac{wl^2}{4}\left(\frac{f_5}{2f_4+f_5}\right) \qquad (24)$$

Slab Coefficients

Figure 8 shows the variation of the terms f_4 and f_5 with values of x. If $x = 0$, there is a uniform moment of inertia and $f_4 = f_5 = 1.0$.

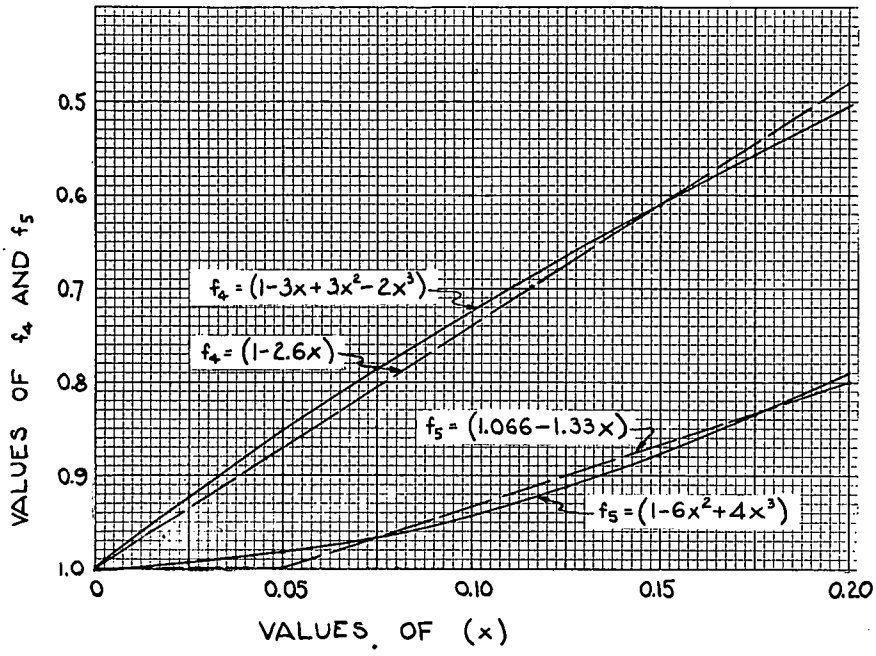


FIG. 8 - SLAB COEFFICIENTS

If the approximate value of x be taken as:

$$xl = b - (b-a)\frac{I}{I_1} \qquad \text{(equation 13)}$$

approximate values of f_4 and f_6 are given as:

$$\begin{aligned} f_4 &= 1 - 2.6x \\ f_5 &= 1.0, \text{ from } x = 0 \text{ to } x = 0.05 \\ f_5 &= 1.066 - 1.33x, \text{ from } x = 0.05 \text{ to } x = 0.2 \end{aligned} \qquad (25)$$

These values are also plotted in Figure 8.

Distribution of a Moment at a Joint

Let joint A be subjected to an unbalanced couple M brought to the beam AB (Fig. 9). If all members have constant moments of inertia, the respective resisting moments are:

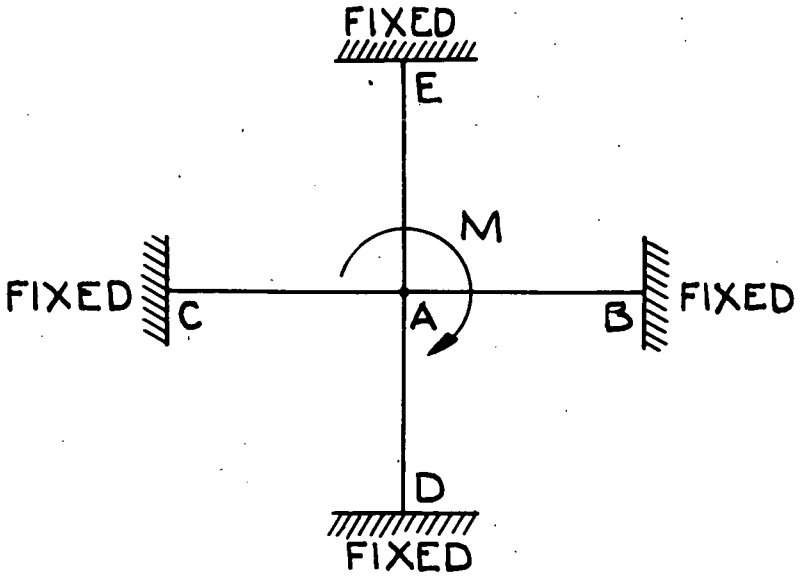


FIG. 9

$$M_{AB} = 4\frac{EI_1}{l_1}\theta_A = 4EK_1\,\theta_A$$

$$M_{AC} = 4EK_2\,\theta_A$$

$$M_{AD} = 4EK_3\,\theta_A \text{ and } M_{AE} = 4EK_4\,\theta_A$$

Also $$M_{AB} + M_{AC} + M_{AD} + M_{AE} = M$$

or $$\theta_A = -\frac{M}{4E(K_1+K_2+K_3+K_4)} = -\frac{M}{4E(\Sigma K)}$$

Then $$M_{AB} = M\left(1\frac{K_1}{\Sigma K}\right)$$

$$M_{ac} = -M\frac{K_2}{\Sigma K}$$

$$M_{AD} = -M\frac{K_3}{\Sigma K}$$

$$M_{AE} = -M\frac{K_4}{\Sigma K}$$

Moment Distribution at Joint of Flat Slab

If the far end of a flat slab span is fixed, the slope of the joint A for the beam AB (Fig. 9) can be determined using equation (18), (19), and (20):

$$\theta_A = \frac{M_{AB}l}{3EI} f_4 - \frac{M_{BA}l}{6EI} f_5 = \frac{M_{AB}l}{3EI} f_4 - \frac{M_{AB}l}{2\times 6EI} \frac{(f_5)^2}{f_4}$$

$$\theta_A = \frac{M_{AB}l}{12EI} \left(\frac{4f_4^2 - f_5^2}{f_4} \right)$$

or $$M_{AB} = \frac{12EI}{l} \theta_A \left(\frac{f_4}{4f_2^2 - f_5^2} \right) = 4EK_1'\theta_A \tag{26}$$

where $$K_1' = \frac{I}{l} \left(\frac{3f_4}{4f_4^2 - f_5^2} \right) \tag{27}$$

then $$M_{AC} = 4EK_2'\,\theta_A$$

The columns can be similarly treated. For the lower column AD, using equations (1), (3), and (10):

$$\theta_A = \frac{M_{AD}h}{3EI} f_1 - \frac{M_{DA}h}{6EI} f_2 = \frac{M_{AD}h}{3EI} f_1 - \frac{M_{AD}h}{2\times 6EI} \frac{f_2^2}{f_3}$$

$$\theta_A = \frac{M_{AD}h}{12EI} \left(\frac{4f_1f_3 - f_2^2}{f_3} \right) \tag{28}$$

or $$M_{AD} = \frac{12EI}{h} \theta_A \left(\frac{f_3}{4f_1f_3 - f_2^2} \right) = 4EK_3'\,\theta_A$$

where $$K_3' = \left(\frac{3f_3}{4f_1f_3 - f_2^2} \right) \frac{I}{h} \tag{29}$$

The upper column is solved using equations (2), (4), and (11).

to give $$M_{AE} = \frac{12EI}{h} \theta_A \left(\frac{f_1}{4f_1f_3 - f_2^2} \right) = 4EK_4'\,\theta_A \tag{30}$$

where $$K_4' = \left(\frac{3f_1}{4f_1f_3 - f_2^2} \right) \frac{I}{h} \tag{31}$$

Now $$M_{AB} + M_{AC} + M_{AD} + M_{AE} = M$$

or $$\theta_A = \frac{M}{4E(K'_1 + K_2' + K_3' + K_4')} = -\frac{M}{4E(\Sigma K')}$$

Then

$$M_{AB} = M\left(1 - \frac{K_1'}{\Sigma K'}\right)$$

$$M_{AC} = -M\frac{K_2'}{\Sigma K'}$$

$$M_{AD} = -M\frac{K_3'}{\Sigma K'}$$

$$M_{AE} = -M\frac{K_4'}{\Sigma K'}$$

Carry-Over Factor

Beams with constant moment of inertia have a moment at the fixed end equal to one-half the distributed beam moment at the joint which has been released.

The moment at the fixed end for flat slabs will be equal to:

$$M_{BA} = M_{AB}\left(\frac{1}{2} \cdot \frac{f_5}{f_4}\right) \qquad \text{by equation (20)}$$

In this case the carry-over factor equals

$$\left(\frac{1}{2} \cdot \frac{f_5}{f_4}\right) \qquad (32)$$

The solution of a continuous flat slab system can be made by the usual procedure of the moment distribution method, if the suitable constants for K' and the carry-over factors are computed. These constants enable one to treat each member as though it had constant moment of inertia.

Illustrative Problem No. 1

To show the application of this theory a solution is given below for a flat slab system of unequal spans. The procedure includes the determination of the necessary constants and a solution by the moment distribution method. More distributions must be made than for systems of constant moment of inertia because the carry-over factors for the beams are so much larger. Figure 10 shows the center-line spans, stiffness values and carry-over factors. Figures 11 and 12 give the necessary computations for the K factors, while Table 1 shows the moment distributions for a uniformly distributed load of 1 kip per foot of length in span FG.

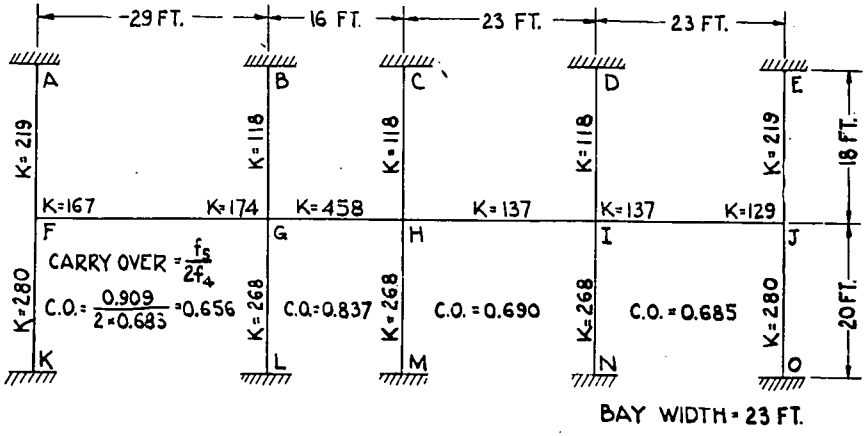


FIG. 10

The fixed end moments for span FG equal:

at F $\qquad$ $\text{F.E.M.} = \frac{wl^2}{4}\left(\frac{f_5}{2f_4+f_5}\right) = -83.4$ ft. kips

at G $\qquad$ $\text{F.E.M.} = +84.0$ ft. k.

By a similar process the moments at the ends of each slab span are successively found for a load of 1 kip per foot on spans GH, HI, and IJ. The results are recorded in Table 2. The dead load moments were figured for a distributed load of 2.7 kips per foot on all spans, and were obtained by adding algebraically the moments for 1 kip per foot at each section considered and multiplying by 2.7.

The live load moments are figured assuming certain spans loaded with a live load of 2.7 kips per foot. In general the maximum negative moment at a given joint is attained, if the two adjacent spans carry live load and then the alternate spans. The maximum positive moment near the center of each span is assumed to occur for a live load on that span and then on each alternate span. These live load moments are also tabulated in Table 2, and the live maxima are added to the dead load moments to give the greatest moment at the ends of each span.

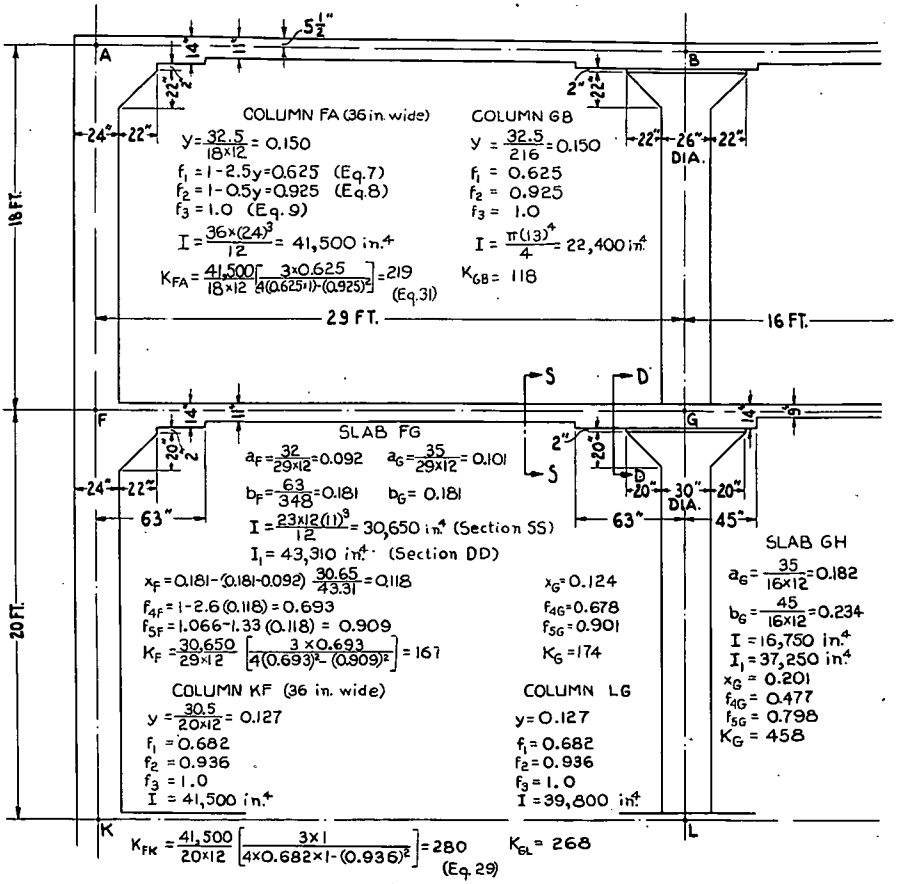


FIG. 11 - SLAB AND COLUMN CONSTANTS FOR SPAN FG

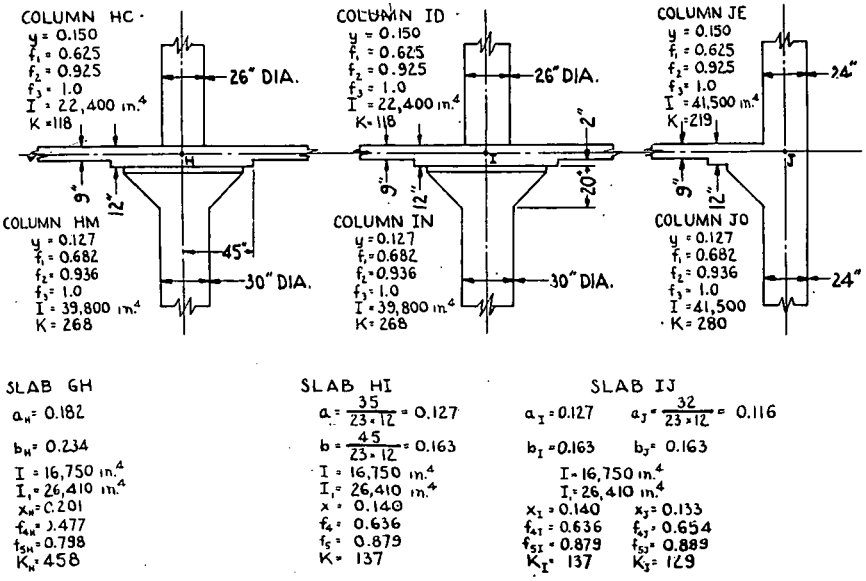


FIG. 12 - SLAB AND COLUMN CONSTANTS

In Figure 13 are plotted the two moment diagrams that give the maximum end moments for the span *FG*. Loading "A" also gives the maximum positive moment.

Correction for Two-Way Slab System

The analyses made above do not allow for the fact that these continuous frames are not one-way slabs. Each slab will act like a plate and deflect laterally as well as longitudinally. Accurate mathematical analyses, such as Dr. Westergaard's, allow for this and the moment coefficients are lower than those given by a one-way analysis. In the case of square slabs and equal spans this reduction amounts to about 28% of the one-way analysis. The accurate analyses are too difficult for use by the ordinary designer, especially with unequal spans.

Mr. Bertin suggests that this reduction can be found by assuming that the critical negative moment is taken at some empirical distance out from the center line of the columns, much as we are accustomed to take the critical negative moment for beams fixed at the

MOMENT DISTRIBUTION (PROB. 1)

	JOINT F				JOINT G					JOINT H					JOINT I					JOINT J		
	FA	FK	FG		GF	GB	GL	GH		HG	HC	HM	HI		IH	ID	IN	IJ		JI	JE	JO
DISTRIBUTION RATIO	0.329	0.420	0.251	0.656	0.171	0.116	0.263	0.450	0.837	0.467	0.120	0.273	0.140	0.690	0.208	0.178	0.406	0.208	0.685	0.206	0.348	0.446
F.E.M.			-83.40		+84.00																	
DISTRIBUTE	+27.44	+35.03	+20.93		-14.37	-9.74	-22.09	-37.80														
CARRY-OVER			- 9.42		+13.70					-31.60												
DISTRIBUTE	+ 3.10	+ 3.95	+ 2.36		- 2.34	- 1.59	- 3.60	- 6.17		+14.78	+ 3.79	+ 8.61	+ 4.42									
			- 1.53		+ 1.55			+12.33		- 5.16					+ 3.05							
	+0.51	+ 0.64	+ 0.38		- 2.37	- 1.61	- 3.65	- 6.25		+ 2.41	+ 0.62	+ 1.41	+ 0.72		- 0.63	-0.55	- 1.24	-0.63				
			- 1.55		+ 0.25			+ 2.01		- 5.22			- 0.43		+ 0.50					-0.43		
	+0.51	+ 0.65	+ 0.39		- 0.39	- 0.26	- 0.59	- 1.02		+ 2.64	+0.68	+ 1.54	+ 0.79		-0.10	-0.10	-0.20	-0.10		+0.09	+ 0.15	+0.19
			- 0.26		+ 0.26			+ 1.79		- 0.85			- 0.07		+ 0.55			+0.06		-0.07		
	+0.09	+ 0.11	+ 0.06		- 0.35	- 0.24	- 0.54	- 0.92		+ 0.43	+ 0.11	+ 0.25	+ 0.13		-0.13	-0.11	-0.24	-0.13		+0.01	+0.03	+0.03
			- 0.23		+ 0.04			+ 0.36		- 0.77			- 0.09		+0.09			+0.01		-0.09		
	+0.07	+ 0.10	+ 0.06		- 0.07	- 0.05	- 0.10	- 0.18		+ 0.40	+ 0.10	+0.24	+ 0.12		- 0.02	- 0.02	-0.04	-0.02		+0.02	+0.03	+0.04
			- 0.05		+ 0.04			+ 0.34		- 0.15			- 0.01		+0.08			+0.01		-0.01		
	+0.02	+ 0.02	+ 0.01		- 0.07	- 0.04	- 0.10	- 0.17		+ 0.08	+ 0.02	+ 0.04	+ 0.02		- 0.02	-0.01	-0.04	-0.02		....	....	+0.01
MOMENT	+31.74	+40.50	-72.24		+79.88	-13.53	-30.67	-35.68		-23.01	+ 5.32	+12.09	+ 5.60		+ 3.37	-0.79	-1.76	-0.82		-0.48	+0.21	+0.27

MOMENT DISTRIBUTION (PROB. 2)

	FA	FK	FG	GF	GB	GL	GH	HG	HC	HM	HI	IH	ID	IN	IJ	JI	JE	JO
DISTRIBUTION RATIO	0.349	0.446	0.205	0.208	0.178	0.406	0.208	0.208	0.178	0.406	0.208	0.208	0.178	0.406	0.208	0.205	0.349	0.446
F.E.M.			-54.50	+54.00														
DISTRIBUTE	+19.02	+24.31	+11.17	-11.21	-9.66	-21.92	-11.21											
CARRY-OVER			- 7.70	+ 7.65				-7.70										
	+ 2.68	+ 3.44	+ 1.58	- 1.59	- 1.37	- 3.10	- 1.59	+ 1.60	+1.38	+ 3.12	+1.60							
			- 1.09	+ 1.08			+ 1.11	-1.10				+ 1.11						
	+ 0.38	+ 0.49	+ 0.22	- 0.45	- 0.40	- 0.89	- 0.45	+ 0.23	+ 0.20	+ 0.44	+ 0.23	- 0.23	-0.20	-0.45	-0.23			
			- 0.31	+ 0.15			+ 0.16	-0.31			-0.16	+ 0.16				-0.16		
	+ 0.11	+ 0.14	+ 0.06	- 0.06	- 0.06	- 0.13	- 0.06	+ 0.10	+ 0.08	+ 0.19	+ 0.10	- 0.03	-0.03	-0.07	-0.03	+0.03	+0.06	+0.07
MOMENT	+22.19	+28.38	-50.57	+49.57	-11.49	-26.04	-12.04	-7.18	+1.66	+ 3.75	+ 1.77	+1.01	-0.23	-0.52	-0.26	- 0.13	+0.06	+0.07

TABLE 1 - MOMENT DISTRIBUTION FOR FLAT SLAB

MOMENT DISTRIBUTION (FIG. 10)

UNEQUAL SPANS (PROB. 1)

	FG	GF	GH	HG	HI	IH	IJ	JI
1 KIP PER FOOT								
ON FG	-72.24	+79.88	-35.68	-23.01	+ 5.60	+ 3.37	- 0.82	- 0.48
ON GH	+ 4.04	+ 7.42	-25.82	+24.85	- 6.04	- 3.63	+ 0.89	+ 0.52
ON HI	- 2.46	- 4.50	+ 15.63	+23.53	-52.60	+48.15	-11.73	- 7.06
ON IJ	+ 0.34	+ 0.64	- 2.21	- 3.46	+ 7.64	+12.26	-49.54	+49.69
DEAD LOAD = 2.7 $\frac{K.}{FT.}$ (ALL SPANS)	-189.86	+225.29	-129.82	+ 59.16	-122.58	+162.41	-165.24	+115.21
LIVE LOAD = 2.7 $\frac{K.}{FT.}$								
ON FG & HI (1)	-201.69	+203.53	- 54.14	+ 1.40	-126.90	+139.10	- 33.89	- 20.36
ON FG, GH, IJ (2)	-183.22	+237.44	-172.02	- 4.37	+ 19.44	+ 32.40	-133.57	+134.27
ON GH & HI (3)	+ 4.27	+ 7.88	- 27.51	+130.63	-158.33	+120.20	- 29.27	- 17.66
ON FG, HI, IJ (4)	-200.77	+205.25	- 60.10	- 7.94	-106.27	+172.21	-167.64	+113.81
ON GH & IJ (5)	+ 11.83	+ 21.76	- 75.68	+ 57.75	+ 4.32	+ 23.30	-131.36	+135.57
MAX. NEG. MOMENT								
$D + L_1$	-391.55	+428.82						
$D + L_2$	-373.08	+462.73	-301.84	+54.79				
$D + L_3$			-157.33	+189.79	-280.91	+282.61		
$D + L_4$					-228.85	+334.62	-332.88	+229.02
$D + L_5$							-296.60	+250.78
DESIGN NEG. MOMENT	-136.17	-196.06	-154.39	-81.33	-102.97	-140.99	-139.52	- 76.37
DESIGN POS. MOMENT	+157.99		+16.92		+81.99		+83.73	

EQUAL SPANS (PROB. 2)

	FG	GF	GH	HG	HI	IH	IJ	JI
DESIGN NEG. MOMENT	-84.61	-137.36	-130.36	-125.36	-125.36	-130.36	-137.36	-84.61
DESIGN POS. MOMENT	+82.24		+77.08		+77.08		+82.24	

TABLE 2 - MAXIMUM NEGATIVE & POSITIVE MOMENTS

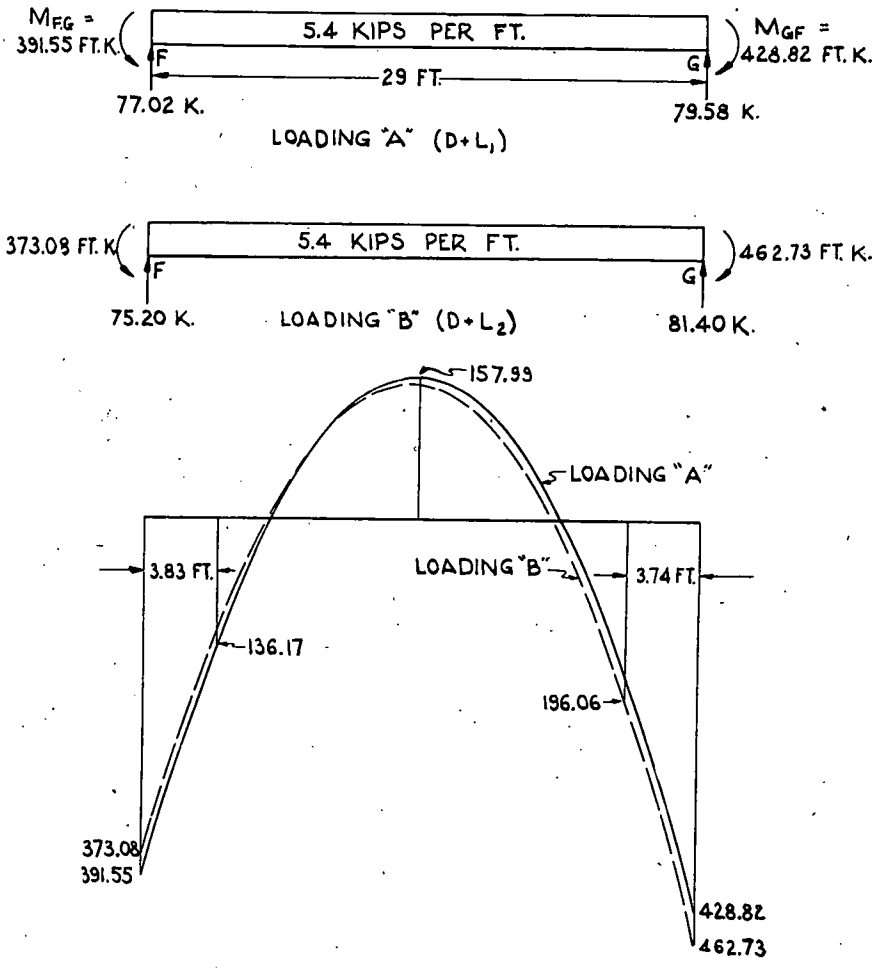


FIG. 13 - MOMENT DIAGRAMS FOR SPAN FG

ends at the face of the column rather than at the center line of the support.

The present A.C.I. and Joint Committee codes only recommend moment coefficients for the case of equal spans with uniformly distributed loads. These coefficients have been used for a number of years and are reasonable values. In this case of equal spans and uniform load is applied to the equations derived above, the negative moment at the center line of each support can be found as illustrated above. Then a moment equation can be written for any section a distance (ml) out from the support. If this equation be equated to the recommended Joint Committee moments, the distance (ml) at which the moment equals the Joint Committee values can be determined. Mr. Bertin finds that this distance is given by the equation:

$$m = 0.50 - \sqrt{0.183 - 0.45a} \qquad (32)$$

where al = distance from column center-line to the edge of the capital, or to the face of the column if there is no capital. This critical section at which the reduced moment equals the Joint Committee value is usually located in the dropped panel somewhat beyond the capital. This equation gives negative moments varying less than 5% from the Joint Committee values.

The maximum positive moment can be taken as found in the original analysis. These values agree closely with those of the Joint Committee for dropped panel slabs but may be as much as 10 to 20% in excess if there is no drop. The greater excess occurs with small capitals.

Neither the A.C.I. or Joint Committee give definite procedure, at present, for unequal spans. Without tests it is problematical whether the approximation of equation (32) can be used to locate the critical negative moment. Until some better *easily applied approximation* is justified, one can *tentatively* use equation (32) to locate the critical section, even though it was derived for the case of fixed ends (zero slope).

Applying equation (32) to the span FG of the illustrative problem of Figure 10, the critical negative moments are found at a distance:

$$\text{(left)} \quad m = 0.50 - \sqrt{0.183 - \frac{0.45 \times 37}{29 \times 12}} = 0.132 \text{ and } ml = 3.83 \text{ ft.}$$

$$\text{(right)} \quad m = 0.50 - \sqrt{0.183 - \frac{0.45 \times 35}{29 \times 12}} = 0.129 \text{ and } ml = 3.74 \text{ ft.}$$

At the critical sections near the columns the negative moment at 3.83 ft. from F equals 136.17 ft. k., at 3.74 ft. from G it equals 196.06 ft. k. The moments used for design for a bay width of 23 ft. would be these negative values and the positive moment of 157.99 ft. k. (Fig. 13). After following a similar procedure for the other spans the design moments are tabulated in Table 2.

A similar procedure can be employed for the spans in the perpendicular direction. These solutions can be used for slabs with or without dropped panel and columns with or without capitals.

Illustrative Problem No. 2

As stated above, the negative moments used for design are determined at a critical section determined by the moments recommended by the A.C.I. and Joint Committee codes for equal spans and uniformly distributed loads. It may be advantageous to check the accuracy of the Bertin method by an illustration. Assume that the bent of Figure 10 consists of four spans of 23 ft. with the column heights unchanged. The flat slabs in spans FG and IJ will be identical with the slab IJ of Figure 12, and the slabs in spans GH and HI will be identical with span HI of Figure 12. The distributed load will again be taken as 2.7 kips per ft. live, and the same amount dead load. A solution similar to that of Problem 1 gives design moments tabulated in Table 2.

These design moments are allocated to the mid and column strips by coefficients given in the A.C.I. and Joint Committee codes. These coefficients are decimals of an empirical moment

$$M_o = 0.09\, Wl \left(1 - \frac{2}{3}\,\frac{c}{l}\right)^2$$

where c = diameter of the column capital. In this problem

$$M_o = 0.09 \times (5.4 \times 23)\ (23) \left(1 - \frac{2 \times 70}{3 \times 23 \times 12}\right)^2 = 177 \text{ ft. } k.$$

Adding the mid and column strip coefficients the total moment for the width of a bay can be computed. The results are tabulated in Table 3. The A.C.I. coefficients are averages for *approximately* equal spans for flat slab floors whose size of dropped panels and column capitals vary relative to the span length. In this particular problem the Bertin method gives moments which are in general about 10 per cent greater than the A.C.I. moments.

SPAN	MOMENT AT	A.C.I.		BERTIN METHOD	
		RATIO OF M_0	MOMENT FT. K.	MOMENT FT. K.	% A.C.I.
FG	EXTERIOR END	0.55	97.3	84.6	87
&	INTERIOR END	0.715	126.6	137.4	109
IJ	POSITIVE	0.44	77.9	82.4	107
GH	ENDS	0.65	115.0	125.4	109
&				130.4	113
HI	POSITIVE	0.35	62.0	77.1	124

TABLE 3 - COMPARISON OF A.C.I. & BERTIN MOMENTS

Distribution of Moment Across Width of Slab

It has been general practice to divide the slab width into two bands, the column and mid strips, and to assume the maximum moments are uniformly distributed across the width of each strip. As noted in Problem 2 the codes recommend the relative magnitudes of the maximum negative and positive moment to be taken by each strip, if the slabs have equal spans. No recommendation is made for distribution if the slabs have unequal spans. The 1925 German Code recommends for all cases a distribution of 75% of the maximum negative moment to the column strip and 25% to the mid strip, which agrees approximately with the A.C.I. coefficients for the interior panels of an equal span bent. The German Code recommends a distribution of 55% and 45% of the positive moment to column and mid strips, which is again in approximate agreement with the A.C.I. coefficients for interior panels.

Such distributions for a bent with unequal spans would necessitate a difficult mathematical analysis involving shear, moment and deflection of the spans of the bent. Such an analysis must be made for each new combination of unequal span and would take too much time. Therefore, until additional experience with this type of construction affords a basis for some simple empirical distribution it seems wise to adopt an arbitrary distribution, such as given in the German Code.

Summary

Adopting a width of one bay, it is possible to solve a flat slab of unequal spans as an elastic frame, obtaining the negative moments on the column center lines. The complete bending moment diagram for the loads adopted can then be constructed. The same procedure can be followed for the spans in the perpendicular direction.

Some combination of live and dead loads will give the maximum positive bending moment for the width of a bay. This moment can be used for design. The combination of live and dead loads which give the maximum negative moment at each column center line can also be determined. These maxima values are too large but the value to be used for design can be obtained from the moment diagram at some critical section beyond the column capital.

The critical moments, positive or negative, are distributed over a width of a bay. It is customary to assume constant moments in each of the two divisions of the bay, the column and mid strips. There is, as yet, no simple method of division for unequal spans and an arbitrary division seems desirable for the time being.

The method of elastic analysis outlined above can also be applied to continuous slabs supported on all four edges by beams or girders. The solution is much easier as the sections of one bay width have constant moments of inertia up to the face of the supporting beam and the columns usually have no capitals or brackets. There is the same problem of the distribution of the critical positive or negative moment between the column and mid strips.

APPLICATION OF PLASTIC THEORY TO THE DESIGN OF MODERN REINFORCED CONCRETE STRUCTURES

By Charles S. Whitney*

(Presented at a meeting of the Structural Section of the Boston Society of Civil Engineers held on April 9, 1947.)

Synopsis

The purpose of this paper is to encourage the immediate and thorough re-appraisal of the general conceptions, principles and theories involved in the design of reinforced concrete structures. The need for construction work in the next few years will be the greatest in the history of the world and it is particularly important at this time to revise design methods so as to produce the most economical structures possible in the light of the best information now available. The reasons for such a revision are outlined and suggestions are made as to how it may be accomplished.

Introduction

Since the publication of the first Joint Committee progress report in 1909 which established the present standard method of beam design, important improvements have been made in the methods of concrete construction resulting in much higher strength and greater reliability. Skill in that field is now so highly developed that engineers may use the material with confidence any place in the world where it is economically practical.

In addition to experience derived from completed structures, a great amount of research work has been done in many laboratories and much information has been gathered regarding the properties peculiar to concrete. This information and experience indicates that, although it has appeared conservative in the past, the standard straight-line theory results in improper control of the factor of safety, in uneconomical designs, and in some cases unsafe structures.

At the time of the adoption of the straight-line theory, it was realized that under a load near the ultimate the stress variation in the concrete of a beam was more nearly parabolic than linear. How-

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ever, it was considered that under design loading the variation could be assumed linear because the stress-strain curve of the concrete was approximately straight up to what appeared to be a reasonable limit of working stress. This relation of course presumed that a plane section before flexure remained plain, stress being proportional to strain, and to distance from the neutral axis. The steel stress was equal to the concrete stress at a corresponding distance from the neutral axis multiplied by n, the ratio of E_s to E_c. Tension in the concrete was neglected. The useful strength of the member was assumed to be limited by the nominal working stress calculated by that theory. Unfortunately, when so calculated, the ratio of design load to actual beam strength bears no particular relation to the ratio of allowable concrete stress to cylinder strength.

The value of n has been the subject of a great amount of research because, while the modulus of elasticity of the steel is reasonably definite, that of concrete is highly variable depending on many factors such as the characteristics of the aggregates, the proportions and age of the concrete and the speed and duration of loading. In order to apply the theory, it was therefore necessary to select empirical values of n which appeared to give the most satisfactory results for concretes of different strengths.

This theory is still standard except that about 17 years ago its application to axially loaded columns was abandoned after extensive tests showed that stresses under load were greatly affected by shrinkage and plastic flow in the concrete which transferred load from the concrete to the steel. It was determined that regardless of what the distribution of stress under working conditions might be, the ultimate strength of the column was approximately equal to the yield strength of the steel plus the area of the concrete cross section times 0.85 f'_c. The allowable load on the column was then fixed by application of factors of safety to the ultimate strength.

Further recognition of the effect of plasticity and shrinkage in concrete was written into the 1940 Joint Committee Report by permitting an arbitrary increase in the stress in steel in the compression side of a beam.

Otherwise still based on the straight-line theory, the present design method is realistic for axially loaded columns, partially so for beams reinforced for compression but completely inconsistent in regard to members subjected to bending and direct stress.

The absurdity of this situation is apparent when it is realized that a member subjected to axial load is treated by the plastic theory; if flexure is added the straight-line theory is used, and if the axial load is then removed, a compromise between the two theories is used. Reconciliation of these three methods is attempted in present codes by specifying a purely empirical formula which determines an allowable stress which varies with the eccentricity of loading.

The writer proposes that one simple and consistent method be used for the entire range of eccentricities from axial load to pure bending.

It should be realized that the science of reinforced concrete design is basically empirical and no theoretical concepts should be permitted to prevent the proper interpretation of test results. The straight-line theory has done just that because while the concept is simple and attractive it results in design formulas which are unnecessarily complicated and inaccurate. A clearer concept of the actual behavior of reinforced concrete members is essential if the material is to be used to best advantage.

The Plastic Theory

The plastic theory will be described here as briefly as possible. It is used to calculate the ultimate strength of a concrete member so that the useful strength can be determined by application of an appropriate design factor.

In the case of a rectangular beam at ultimate load, it assumes that the compressive stress in the concrete can be represented by a rectangular stress block of uniform intensity equal to 0.85 f'_c. Tension in the concrete is neglected.

The steel is assumed to work at yield stress in tension if the beam is not over-reinforced for tension. Compression reinforcement also works at yield stress in compression because the strain in the concrete when compression failure occurs is greater than the steel strain at yield stress.

These assumptions result in the simplest possible equations based entirely on f_s and f'_c without consideration of the value of n.

The basic geometry of the plastic theory is indicated in Fig. 1. There is evidence that the actual stress distribution under maximum load applied at ordinary testing speed may be somewhat as shown in the upper left hand drawing. This is based on the assumption that

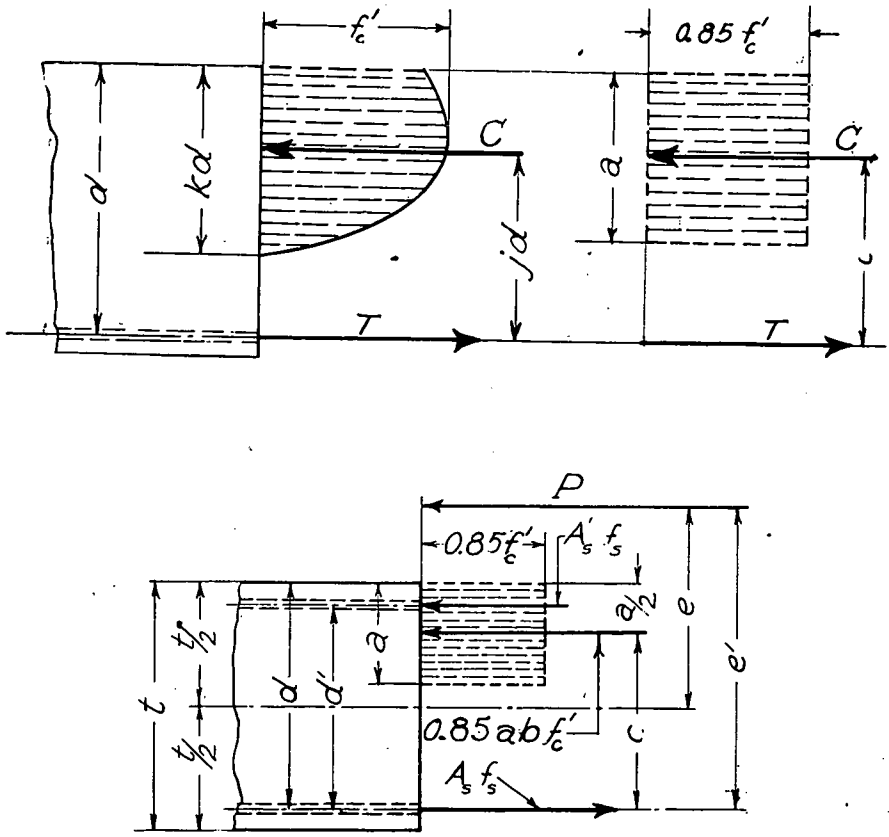


FIG. 1

the curve of stress variation in the beam is similar in shape to the stress-strain curve from zero strain up to the maximum strain in the concrete of the beam at failure. This latter is much greater than the strain in a cylinder at maximum load. However, the shape of the stress distribution curve and the location of the neutral axis are unimportant because tests indicate that the compressive stress can be represented by a rectangular block whose area is $0.85\ af'_c$. When the plastic theory was developed the average intensity of stress was taken as $0.85\ f'_c$ because it was consistent with the value determined by tests of columns under axial load, but subsequent research indicates a basic reason why it is correct.

A study of the stress-strain characteristics of concrete shows that during the rapid change which takes place in stress distribution in a beam during failure it may actually pass through a phase closely resembling the rectangular stress block with the maximum stress equal to 0.85 f'_c. In the neighborhood of maximum load, a great change occurs in the stress distribution without much change in moment.

The stress-strain relation of concrete is greatly affected by the speed and duration of loading as well as the age of the concrete and varies somewhat, of course, from one sample to another. This is caused by plastic flow which is a function of time. Instead of one stress-strain curve for any particular concrete, a different curve will be found for each different speed of loading and it is therefore necessary to represent the stress-strain relation by a family of curves such as shown in Fig. 2. This is based on the reports of numerous tests. The standard rate of loading for cylinder tests is 2000 p.s.i. per minute. Tests reported by Jones and Richart[1] show a logarithmic relation between cylinder strength and rate of loading. Cylinders

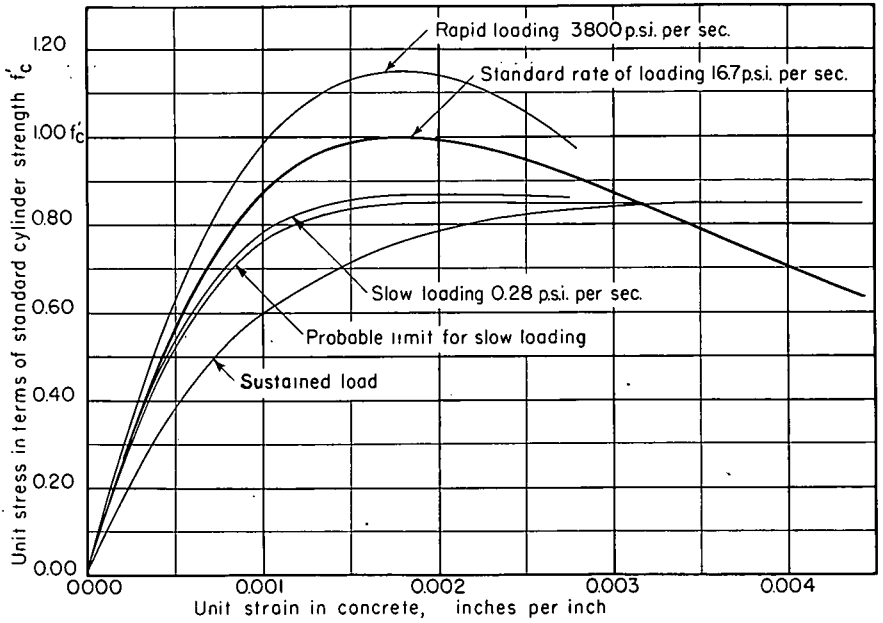


Figure 2 Stress-Strain Relations for Concrete Cylinders

broken in about one second showed about 15 per cent higher strength than the standard rate. When loaded slowly, at less than one half a pound per square inch per second, the strength dropped to about 87 per cent of the standard, indicating a limit of about 85 per cent for very slow loading.

The fact that there is a descending branch of the stress-strain curve as shown in Fig. 2 has been demonstrated by a number of tests. The sudden failure of cylinders in a testing machine does not indicate that the stress-strain curve drops abruptly but is due to the spring action of the testing machine. (See discussion of reference 2, page 584-2 ACI Proceedings, Vol. 39).

Recent tests made by the Bureau of Reclamation showed that failure may be caused by sustained loads lower than the strength indicated by the standard test. It was found that 96 per cent of the standard test load caused failure in 3 to 8 minutes, 92 per cent in 25 minutes and 88 per cent in 235 minutes. An extrapolation of the data indicated that a load of about 85 per cent might be sustained indefinitely. This agrees with the tests reported by Jones and Richart.

It has also been demonstrated that plastic flow not only causes progressive increase in strain when stress is sustained but it conversely causes a progressive reduction in stress if the strain is maintained constant.[3-4] This permits a readjustment of stresses in a beam and a rapid reduction of the maximum stresses as the maximum load is approached. The rate of plastic flow increases greatly at very high stresses.

These stress-strain characteristics all point to the fact that stress distribution in a beam at failure must actually closely approximate the rectangular stress block with a small and insignificant parabolic variation next to the neutral axis. The stress over a greater part of the compression area must be about 0.85 f'_c because a higher stress cannot be sustained and is relieved by plastic flow. The stress distribution curve may then be represented by the limiting curve of Fig. 2 for slow loading but at that stage the maximum strain at the compression face will be considerably greater than it was when the distribution followed the curve indicated in Fig. 1. This redistribution of stress can occur without much change in bending moment.

Glanville and Thomas tested beams under sustained load and reported as follows:[5] "The ultimate strength of a reinforced concrete beam is by no means impaired by the strain changes that occur under

prolonged loading. Taking an average for the three series, it will be seen that the strength of the beam submitted to prolonged loading is exactly the same as that of the beam which was maintained under no load and tested at the same age as the main beam. This important result is, as in the case of columns, due to the very large inelastic movements which occur in both the concrete and the steel when stresses are high. These movements lead directly to readjustments in the stresses both in tension steel and the compression concrete, which largely compensate or eliminate the effects of initial stresses due to previous loading."

In an under-reinforced beam, the total compression is equal to the yield strength of the steel. This condition determines the height of the stress block, a, and the steel lever arm, c. When the yield stress of the steel is reached, the stretching of the steel cracks the beam reducing the area of concrete in compression until it is small enough to permit the concrete at the top of the beam to crush. This reduces the steel lever arm and causes progressive failure. The limiting values for balanced reinforcement are determined empirically.

The corresponding relations for bending and direct stress are shown in the lower part of Fig. 1.

In the case of flexure of a beam reinforced for tension only, this theory results in the following equations:

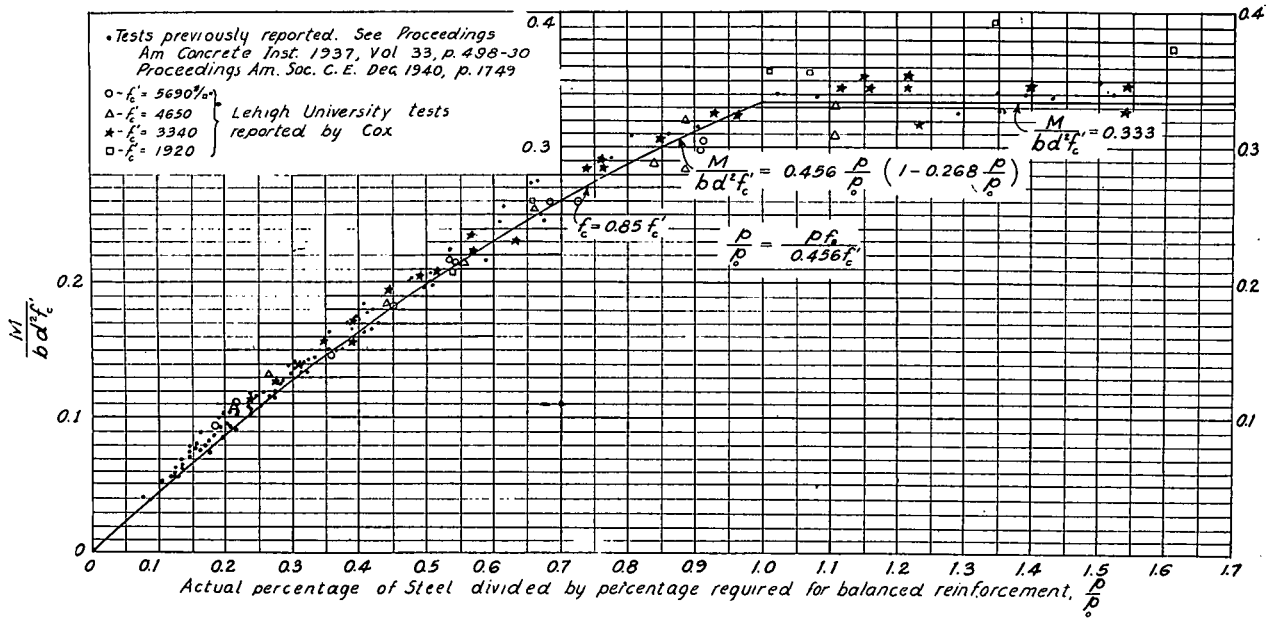
For under-reinforced beams

$$\frac{M}{bd^2} = pf_s\left(1 - \frac{pf_s}{1.7\,f'_c}\right) \tag{1}$$

which for balanced reinforcement or an over-reinforced beam, reaches a maximum value (determined empirically from tests) of

$$\frac{M}{bd^2} = \frac{f'_c}{3} \tag{2}$$

The correlation between these equations and tests of beams is shown in Fig. 3. This correlation is much better than can be obtained with the standard straight-line formula. For under-reinforced beams the latter gives values of steel lever arm which may be more than five per cent too low and for balanced reinforcement thirteen per cent too high. This results in lack of economy in under-reinforced beams and reduction of factor of safety in heavily reinforced beams and is one of the reasons why the use of the straight-line theory with



THEORETICAL ULTIMATE STRENGTH OF CONCRETE BEAMS COMPARED WITH RESULTS OF TESTS

FIGURE 3

higher allowable concrete stresses would not be as satisfactory as the plastic theory.

The simplicity of these equations involving only f'_c and f_s without the use of the variable, n, is obvious when compared with the straight-line equations. It is also indicated by Fig. 4 which gives a universal design chart for rectangular beams, covering all values of f_s and f'_c.

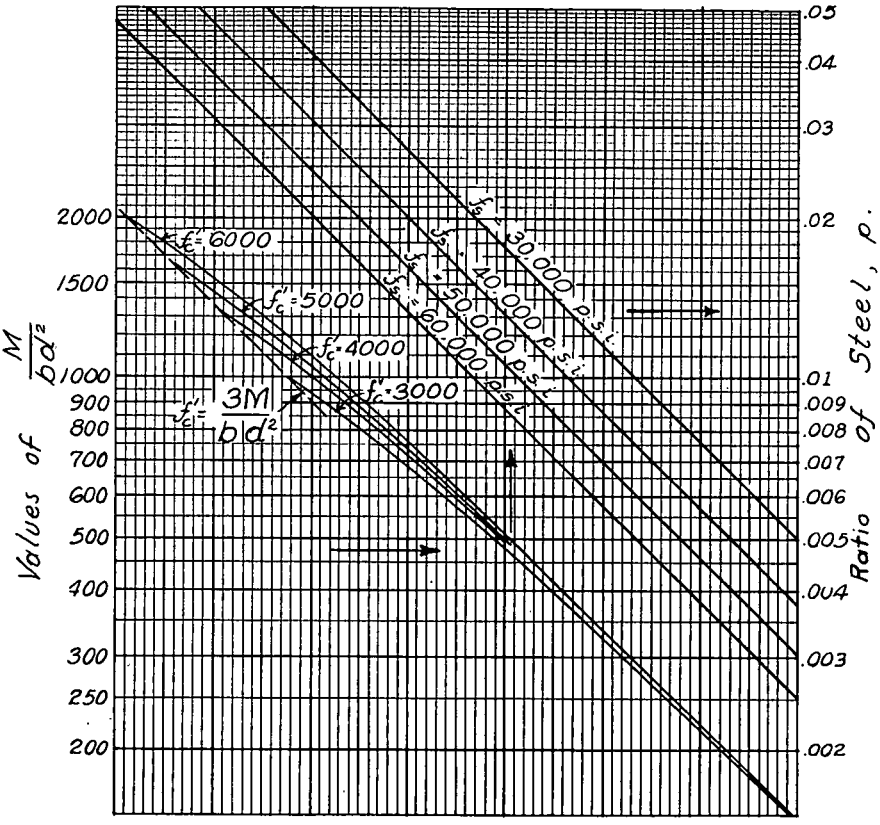


FIG. 4

ULTIMATE BENDING MOMENT AND PERCENTAGE OF STEEL FOR RECTANGULAR BEAMS

Based on Equations 1 and 2

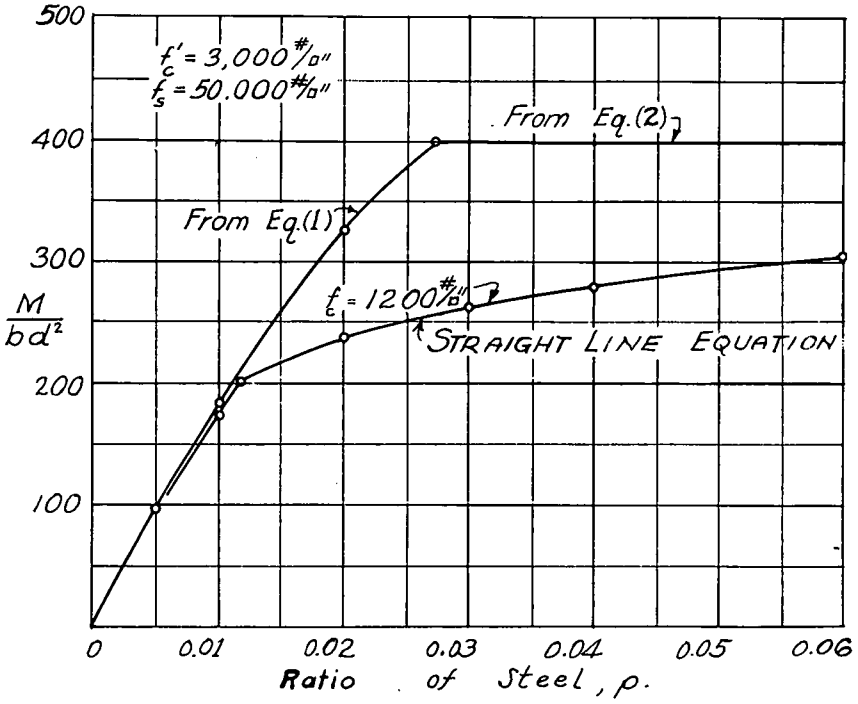


FIG. 5
COMPARISON OF RESISTING MOMENT OF RECTANGULAR BEAMS CALCULATED FROM STRAIGHT LINE EQUATION AND PROPOSED FORMULAS WITH FACTOR OF SAFETY OF 2½.

Fig. 5 gives a comparison between the resisting moments of rectangular beams computed from the straight-line equation with the allowable concrete stress equal to 1200 p.s.i. and the plastic theory with $f'_c = 3000$ and a factor of safety of 2½. It is seen that the strength of a rectangular beam as controlled by the concrete strength is very much greater than is recognized by the straight-line equation. An attempt to reconcile the straight-line equation with actual beam strength would result in allowable working stresses which would appear absurdly high, so high in fact that the allowable stress would no longer be limited to the straight portion of the stress-strain curve,

and working stress calculations would have even less significance than they do with the present limits.

A very valuable investigation of the plastic theory has been made by R. H. Evans,[6] who tested 41 beams with concrete strengths varying from 975 to 7250 p.s.i. and steel ratios from 0.0047 to 0.067 which showed excellent agreement with the writer's formulas. His paper also includes a comparative study of other theories which have been proposed since 1914.

Perhaps the greatest advantage of the plastic theory is that it provides a simple and consistent treatment for members under bending and direct stress for the full range of cases from axial load to pure flexure. The centrally loaded column and the beam under pure bending are only special cases of the general problem. The treatment of members under bending and direct stress has become extremely important because most concrete structures are now designed as continuous frames. Frame analysis methods are now in general use but the standard method of design of the members after moments and thrusts are calculated is very unsatisfactory. It is unnecessarily complicated and does not permit the proper control of the factor of safety which should be the objective of all design. The plastic theory permits proper control in the simplest possible manner.

The ultimate strength of a rectangular member with symmetrical reinforcement is expressed as follows:

When the concrete strength controls

$$P = \frac{2A'_s f_s}{\frac{2e}{d'} + 1} + \frac{btf'_c}{\frac{3te}{d^2} + 1.178} \qquad (3)$$

For greater eccentricity when the tensile strength of the steel controls

$$P = 0.85\, bf'_c t \left\{ \sqrt{\left(\frac{e}{t} - 0.5\right)^2 + \frac{d'}{t} p_t m} - \left(\frac{e}{t} - 0.5\right) \right\} \qquad (4)$$

The correlation between these equations and a series of tests by Bach are shown in Fig. 6 which is plotted with direct load, P, as the ordinate and the bending moment, Pe, as the abscissa. This presents the full range of eccentricity from zero to infinity in one quadrant and permits consideration of the effect of any combination of thrust and moment.

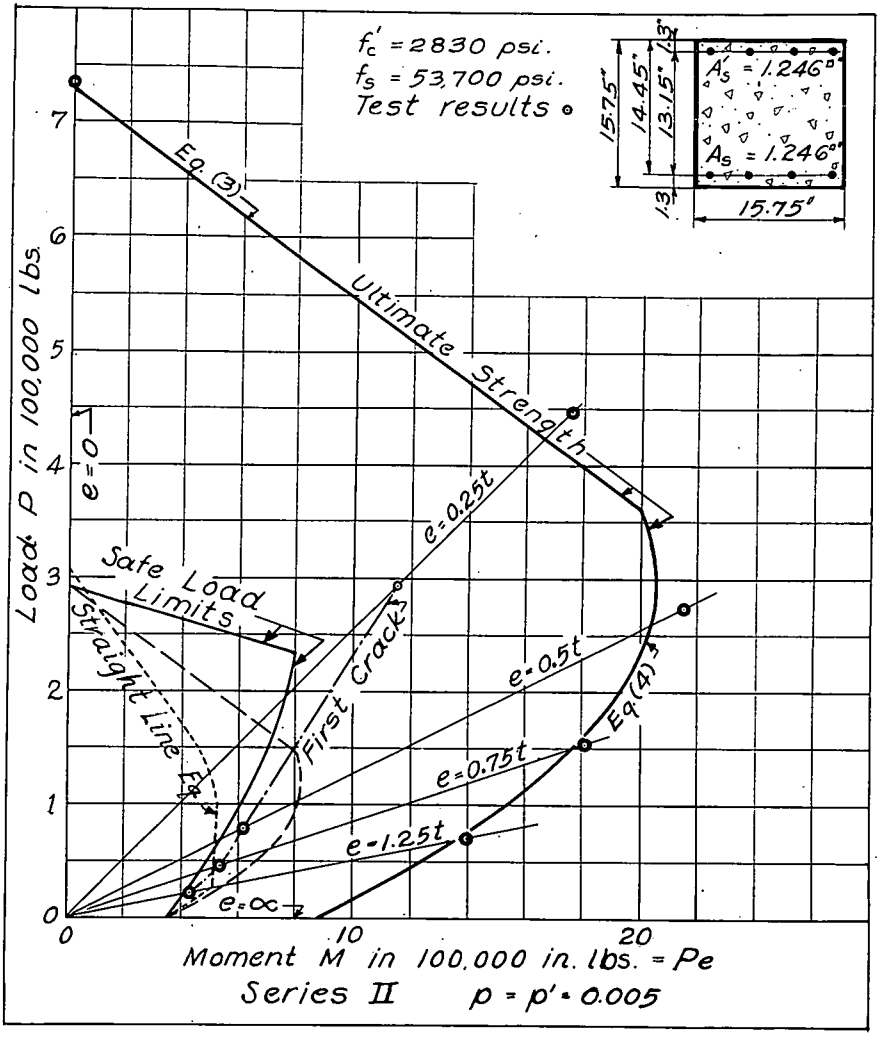


FIG. 6

The upper or compression control line plotted from Eq. 3 is a straight line connecting the point on the Y-axis representing the strength of the axially loaded member with the point on the X-axis corresponding to the flexural strength of the member as controlled by the strength of the compression side assuming sufficient tension steel to develop it.

The lower or tension control line plotted from Eq. 4 is a curve which reaches the X-axis at the point representing the flexural strength of the member as controlled by the strength of the tension steel.

Any point lying in the area to the left of the two control lines represents a combination of load and moment which the member will carry without failure, but if the point lies to the right of a control line failure will result.

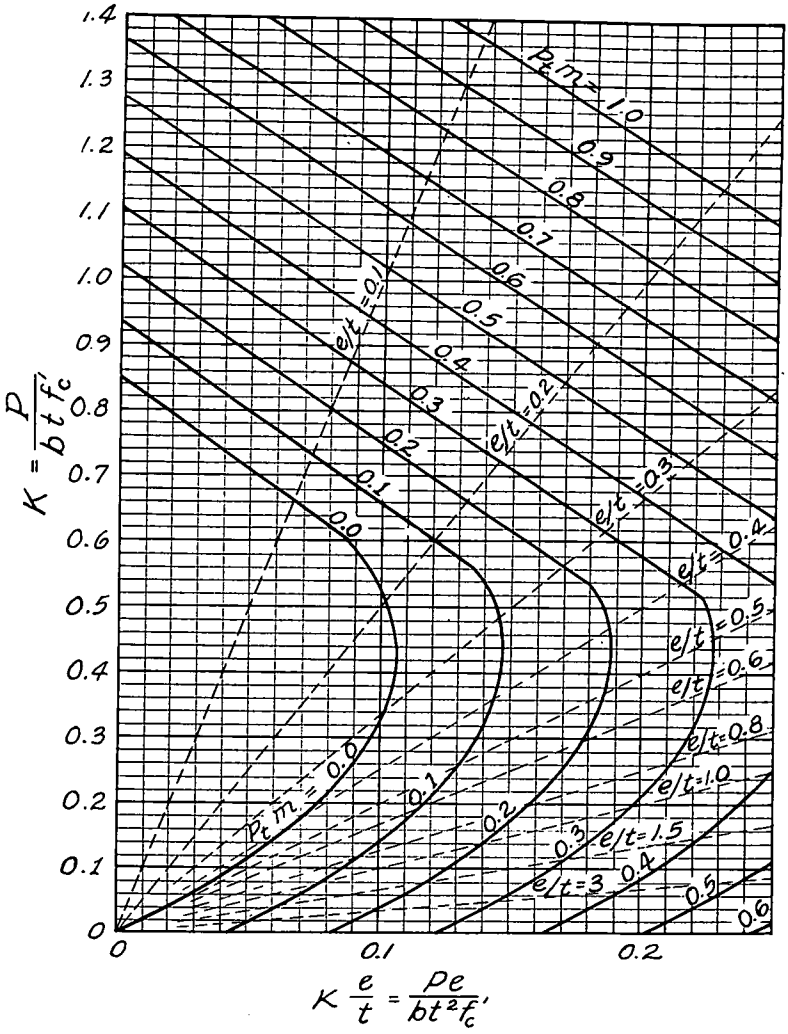
It is particularly important to note that when the bending moment is greater than the beam strength of the member (the intercept of the tension control curve with the X-axis), failure may be caused by a reduction of the direct load without a change in bending moment. For the greater part of the tension control curve, the ultimate moment increases with increase in direct load.

It should also be noted that the standard straight-line method is shown by the dotted line in Fig. 6 which bears no consistent relation to actual strength.

The ordinary method of design using allowable working stresses is very inadequate when applied to members under bending and direct stress. In the first place, it assumes that in case of over-load both thrust and moment increase proportionally with no resulting change in the eccentricity. In most structures, there is no probability of such a condition because the thrust is usually produced to a greater extent by dead load and the bending by live load. This is particularly true in the case of arches. A critical condition may therefore be produced by increase in the live load without change in dead load.

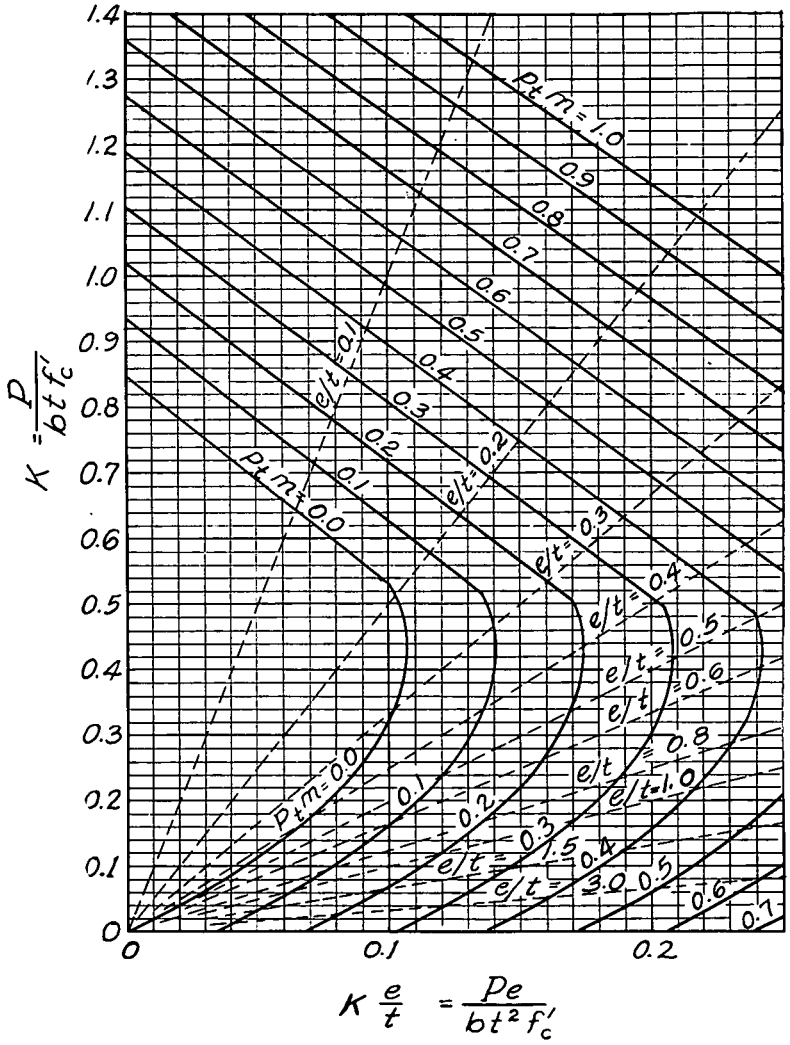
By means of charts made from Eqs. 3 and 4, such as those in Figs. 7, 8 and 9, a suitable factor of safety under any possible combination of thrust and moment may easily be provided. These give the ultimate thrust and moment for a symmetrically reinforced rectangular member corresponding to any values of f_s, f'_c and p. Separate charts are needed for different values of $\frac{d}{t}$. The value of p_t is the total steel area divided by the total concrete area and $m = \frac{f_s}{0.85\, f'_c}$.

Ultimate strength methods of design based on the plastic theory have been in use in several countries for some years. The most complete and interesting of these methods is probably the Russian method adopted in 1937 based on an extensive program of research.



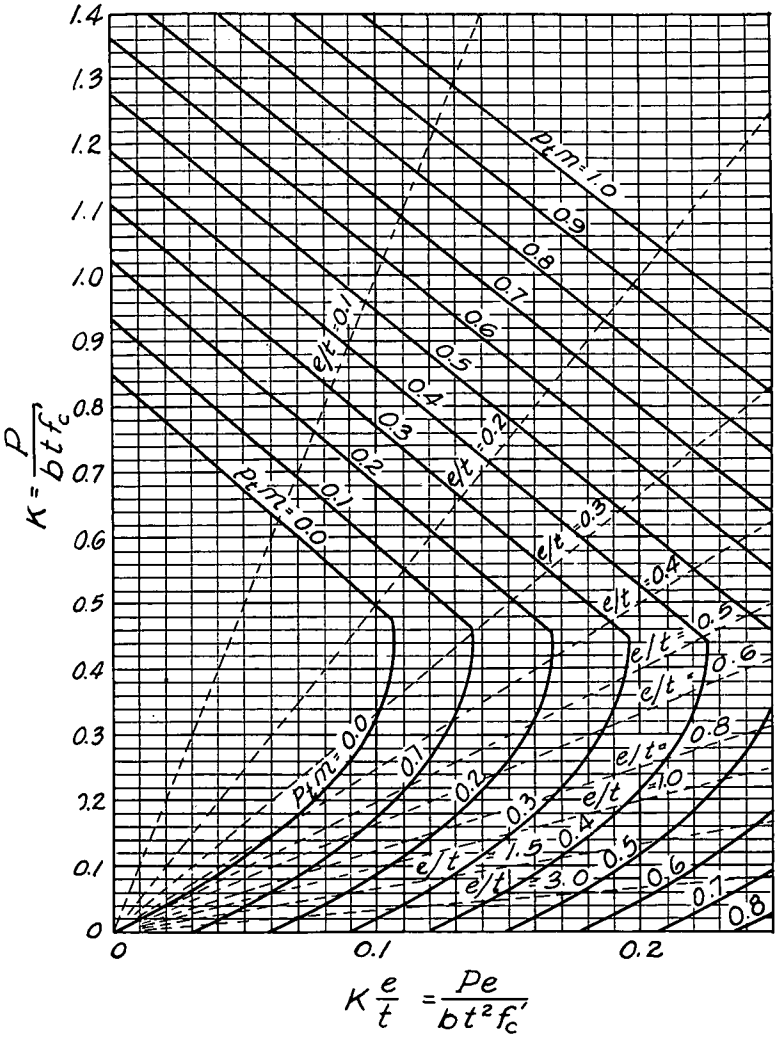
Ultimate Strength
of Rectangular Section
Symmetrically Reinforced
$d = 0.95t$

FIG. 7



Ultimate Strength
of Rectangular Section
Symmetrically Reinforced
$d = 0.9t$

FIG. 8



Ultimate Strength
of Rectangular Section
Symmetrically Reinforced
$d = 0.85t$

FIG. 9

Russian Design Specifications

The following notes on the Russian design method are translated for the writer by Mr. P. T. Chulkoff from "Reinforced Concrete Structures" by K. V. Saknovsky printed in the Soviet Union, 1946 edition.

The two basic Russian formulas for ultimate strength are:

For columns $$P = f_c A_c + f_s A_s \tag{5}$$

For beams $$M = bd^2 p f_s \left(1 - \frac{p f_s}{1.89 f_c}\right) \tag{6}$$

in which f_s = yield strength of steel

f_c = ultimate strength of concrete

according to the following table.

Table 1

The ultimate compressive strength of 8" cubes, p.s.i.	4970	4270	3550	2840	2420	1990	1560	1280	995	710
The ultimate compressive strength of concrete for columns, p.s.i.	3200	2840	2490	2060	1780	1530	1250	1025	797	570
The ultimate compressive strength of concrete for beams, slabs, etc., p.s.i.	3980	3550	3130	2560	2200	1920	1560	1280	995	710

It will be noted that Equation (6) is in the same form as Equation (1) with a slightly different numerical constant. Since both equations are empirical the reason for this difference is interesting. The Russian equation is based on the assumption that the stress variation in flexure follows a cubic parabola as in the method proposed by L. J. Mensch in 1914,[6] but f_c has an empirical value which varies with the grade of concrete. Seven of the ten standard Russian grades of concrete listed in Table 1 have cube strengths below 3000 p.s.i. It is apparent therefore that while the method proposed by the writer is intended for use with concrete cylinder strengths of

3000 p.s.i. or more, the Russian method is intended for application to much lower grade concrete and this rather complicates the problem of selecting the empirical constants.

For cases of combined bending and direct stress, the same parabolic stress distribution is assumed and equations are derived in which the empirical values of f_c are used for the various grades of concrete.

For design purposes the Russian code provides for a factor of safety or "coefficient of reserve" which depends on various conditions. This is explained by Mr. Chulkoff's translation as follows:

"The established coefficients of reserve of strength (k) depend also upon the character of loadings (the possibility of overloading of structure); therefor "Standards" for 1943 and 1945 consider the value k as being dependent upon the ratio of stresses caused by temporary loadings (S_t) and the stresses caused by permanent loadings (S_p); hydrostatic pressure pertains to the latter loading. It is clear that the coefficient of reserve should be larger for the cases where the temporary loading is predominant. The mentioned dependance is accepted for the basic as well as the basic and additional effects. Only for special effects, where S_t always is predominant, there is no reason to consider the coefficient k as being dependent upon the ratio $\frac{S_t}{S_p}$; taking into consideration that these effects occur very rarely, the coefficients k are given at considerably lower values.

"The values of coefficients k depending upon the ratio $\frac{S_t}{S_p}$ and various effects are given in Table 2.

"The choice of the coefficient of reserve, k, depends upon the ratio of stresses and not upon the loadings that cause the stresses. This makes it possible to compare the stresses resulting from loadings of various configurations.

"However, we should not complicate the design by blindly following the letter of the standard. For example, in the design of a statically indeterminate beam, it should be understood that we would not use different coefficients of reserve for various sections, although the ratio $\frac{S_t}{S_p}$ may differ quite considerably. For eccentrically compressed members it is recommended to use the coefficient of reserve based on the ratio of moments and not on the ratio of direct forces,

TABLE 2.—COEFFICIENTS OF RESERVE k.

Cause of Failure		Considered Effects						
		Basic ratio $\frac{S_t}{S_p}$			Basic and Additional ratio $\frac{S_t}{S_p}$			
		Less than 0.5	Between 0.5 and 2.0	More than 2.0	Less than 0.5	Between 0.5 and 2.0	More than 2.0	Special
Reinforcing steel reaching yield point or concrete reaching its limit of strength in compression	For main members which are subjected to compression primarily (Columns, Arches)	1.8	2.0	2.2	1.6	1.8	2.0	1.4
	For other members	1.6	1.8	2.0	1.5	1.6	1.8	1.3
Concrete reaching its limit of strength in tension (main stress)		2.0	2.2	2.4	1.8	2.0	2.2	1.6

because the increase in eccentricity is more dangerous than the increase of the direct load with constant eccentricity."

The last sentence is a recognition of the point emphasized by the writer.

In Table 2 it appears probable that in the case of the "basic ratio" S_t is the stress caused by live or temporary load plus impact while in the case of "basic and additional ratio" the effect of temperature variations and other secondary influences are included. The column headed "Special" applies to very unusual and extreme loading and probably contemplates such things as earthquake effects or temporary construction equipment loading.

There is a further provision that for pre-fabricated units which are subject to better control than ordinary construction, the coefficients in Table 2 may be reduced by 0.2 and for transportation and erection stresses the coefficient of reserve may be 1.8 for tensile stresses in the concrete or 1.5 for all other stresses.

Brazilian Design Specification

The Brazilian code permits the use of the plastic theory ultimate strength method as an alternate to the straight-line theory with allowable working stresses under the following conditions.

In structures not subject to vibrations or shock, it is permitted with a factor of safety not less than two, that flexural members be proportioned according to ultimate strength assuming the steel stress to be equal to the yield strength and the concrete stress a rectangular block with average stress equal to three-quarters of the cylinder strength.

This results in the following equation for beams:

$$M = bd^2pf_s\left(1 - \frac{pf_s}{1.5\,f'_c}\right) \tag{7}$$

Factor of Safety

The objective of structural design is to keep actual loads within the useful strength of all members and to provide a suitable margin of safety against collapse due to accidental overload. The design load should not crack the structure seriously nor otherwise impair its usefulness, and accidental overload should not exceed the ultimate strength.

The design load should be the greatest load or the most unfavorable loading condition that may reasonably be expected in service. The margin of safety which should be provided will depend on the nature of the loading, on its duration, frequency, impact, and the accuracy with which it can be estimated. A smaller factor may suffice for dead than for live loads.

It is obvious that such control of the factor of safety requires accurate knowledge of the actual strength which cannot be obtained from the standard straight-line equations.

Proper control of the factor of safety also demands that design formulas giving the strength of reinforced concrete members be based on definite concrete strength. The formulas should not contain a factor to cover the variability of the strength of the material. In the case of steel, the material is supplied under a rigid specification and it is customary to assume a definite strength in design. In the same way, a definite concrete strength can be obtained and should be assumed for design purposes. The manufacture of concrete which

will have a dependable minimum strength is a distinctly separate problem and should not be given any consideration in the design formulas. Separate consideration will focus attention on the factors involved in control of concrete strength and result in better work while coverage of the variability by a blanket factor of safety in design will encourage careless work. When a definite concrete strength is assumed for design of a structure, the concrete mix should be designed for a higher strength with sufficient margin of safety to assure satisfactory minimum strength. This margin will depend on the accuracy of control which will be exercised on the job. A well managed job should not be penalized because many jobs are poorly controlled. If the control is so poor that the proper margin of safety in the mix design cannot be estimated, concrete should not be used as a structural material.

It is probably not generally known that on well controlled jobs the variability of concrete strength is not very different from that of the steel. If a series of cylinders is tested on a job, the standard deviation of the compressive strengths divided by the average strength is a good index of the variation. The following table taken from the "Concrete Manual" of the Bureau of Reclamation gives the per cent variation on various Reclamation Bureau work.

TABLE 3

Place of test	Concrete cast for	Average compressive strength, p.s.i.	Number of tests	Standard deviation	Variation per cent
Denver	Laboratory investigations	3280	27	218	6.7
Field	Large dam	5380	170	400	7.5
Field	Medium size dam	4200	39	350	8.3
Field	Canal lining	4170	13	360	8.7
Field	Small structures	4340	21	540	12.6

In comparison with this, 27 tests on reinforcing steel on a recent bridge job gave a variation in yield strength of 8.19 per cent. The maximum was 58,630 p.s.i. and minimum 43,880 p.s.i. and the specified minimum 40,000 p.s.i The variation in yield strength shown by tests of the silicon steel for the towers of the Golden Gate Bridge was 7.88 per cent. In the design of steel structures, it is assumed that the steel will have specification strength and the same assumption

should be possible in the case of properly controlled concrete structures.

Need of Revision of Standard Code

The full value of reinforced concrete as a construction material cannot be realized until there is available for its design a method based on the combined experience gained in the laboratory and on actual structures. Such application of recently gained knowledge will indicate new possibilities in the use of the material which have not yet been generally recognized. Concrete should take its place with the other new materials whose development has been stimulated by the war and whose possible post war applications appeal so greatly to the imagination.

The theory of elastic frames is now widely used for the analysis of reinforced concrete structures and many designers have also recently become skilled in its use in the aircraft industry. These designers have also become familiar with the use of ultimate strength formulas and load factors in that industry. The principal need of reinforced concrete designers today is a set of simple formulas which will predict the strength of members so that appropriate and consistent factors of safety may be used.

Design formulas should be based on the actual strength of reinforced concrete members and on a true concept of the action of the material under load. The standard "straight-line" formulas do not satisfy either of these conditions. The claim is made that they can be used to calculate the unit stresses with fair accuracy under safe working loads but that is true only to a very limited extent. They do not indicate true steel stresses and steel stresses are important because a flexural member with an economical amount of steel will usually fail in tension. They do not indicate the true stress in the concrete because it is greatly affected by shrinkage and plastic flow. They will not consistently indicate what the safe useful strength of a member may be.

In order to make progress in the design of concrete structures, it will be necessary to clarify in the minds of both designers and research men the true action of the material. The concept of ideal elastic behavior has not been helpful in that direction insofar as the strength of the members is concerned although it is as yet the best tool for analyzing continuous frames. Great effort has been expended study-

ing the modulus of elasticity of concrete in order to determine the value of n, but it now appears that n need not be used in formulas for the strength of members. It is possible to predict the strength quite accurately with simple formulas involving only the geometry of the member, the cylinder strength of the concrete and the yield strength of the steel.

The writer has had occasion to investigate the strength of many old reinforced concrete structures which were not designed according to the Joint Committee method nor any other standard method. In any case, the Joint Committee formulas do not give a good indication of the actual safe strength.

To mention one detail, the standard method requires greater depth at the support of a continuous beam that can be justified by the actual strength of the beam in compression. Because of a general understanding that concrete has considerable excess strength, it has been quite common practice among designers to neglect high nominal stresses at that point and omit deepened haunches or compressive reinforcement which would be required by adherence to the code.

The Joint Committee method of design establishes unnecessary limitations which result in less economical and less satisfactory structures than can be designed by a more realistic method.

Standards should be given constant scrutiny and should be revised as frequently as may be justified by new developments. The inertia encouraged by the infrequent Joint Committee reports freezes standard practice, discourages research and prevents progress. A realistic re-appraisal of basic principles at this time will create new interest and stimulate much needed research.

Rather than wait some years until the complete application of the plastic theory can be perfected, its use to the extent now justified by present knowledge should be permitted as an alternate to the present method. This would have immediate benefit and permit the gradual adoption of improved methods without the difficulties which would be caused by a radical mandatory change.

It should be noted that the reasons for recommending the adoption of the plastic theory are practical not theoretical and that it has better support from laboratory tests and behavior of actual structures than has the straight-line theory. No matter what theory is used much research is still needed to permit further refinement in design.

The use of the plastic theory will have the effect of reducing the

size of concrete members and to take full advantage of its possibilities it will be necessary to revise thoroughly the methods of design for bond and shear stresses. This will require basic research to determine the effect of the newly available hi-bond reinforcing bar which should result in considerable economy. A study should be made of the effect of shear in combination with bending and with direct stress and bending. Additional information is needed on the strength of members which are other than rectangular in form and on the effect of bending on diagonal axes.

Conclusion

Reinforced concrete is still a new material which should be treated with imagination and without prejudice. Any real improvement in the efficiency of its use will be very important during the next few years when more structures will be needed than have ever before been built during any one age. Because American engineers have lagged behind those in other countries in the adoption of progressive ideas in design, provision should be made in design codes permitting the use of well substantiated new methods.

Concrete will have an important use in the construction of long span hangar roofs with arches so slender that they must be designed to prevent buckling due to deflections. This requires the determination of the ultimate strength of the member as in aircraft design and the standard working stress method is entirely unsatisfactory. The use of the plastic theory will also result in considerable economy in the design of long span parking garages where the live load is comparatively light and dead weight is an important factor.

The method proposed will permit the design of lighter, more economical and more serviceable reinforced concrete structures. It will improve the position of that material in competition with structural steel, particularly in the field of long span structures. The method for design of reinforced concrete members should be considerably simpler than that now used, saving time in design and encouraging more attention to the analysis of elastic frames.

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Discussion by Raymond C. Reese*

Mr. Charles S. Whitney's paper on the application of plastic theory to reinforced concrete design is a very timely and important one. While I am interested in seeing this proposal carried out, there are some points that would need further clarification before practical use could be made of the method. The following comments approach the problem from divergent angles under such headings as general philosophy, safety factors, applicability of tests, diagonal tension and bond, plastic sagging, bending and direct stress, relative economy, and long-span structures.

A whole new philosophy or concept of structural design is introduced by this method. At present the designer uses certain loadings, stresses, moment factors, impact allowancess, and similar items with the mental attitude that he is keeping his probable stresses within certain safe working limits under fully-loaded conditions. His experience and the manner in which his structures have performed are a guide to his judgment as to how he should vary these different factors to obtain satisfactory results. Good design is something more than the solution of standard formulas. A very considerable amount of common sense and judgment goes into the selection of the values which are introduced into the formulas. Mr. Whitney's proposal changes the designer's viewpoint. Under it, he endeavors to predict outright failure at some multiple of the probable dead, live, impact, and other loads. His thinking would be diverted from the channel of staying "on the safe side," to theoretical failure under multiple loading. This might be a considerable improvement; but, in any event, it would change the designer's entire concept. He would have to revise his experience and learn from actual structures how to vary the different factors to obtain safety and economy.

Professor Van den Broeck's proposal of limit design in structural steel to some extent parallels Mr. Whitney's plastic design for concrete. They give the designer greater scope in adjusting the various contributing factors.

It will be possible to apply different multipliers to the dead load (which is determinable to a fair degree of precision) and to the live load (which is usually quite variable and which is the item that would change if the occupancy of the building changes).

The second point to be considered, then, is what safety factors

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or multipliers to apply to the dead and live loads. Since the dead load can be fairly precisely determined and is not likely to vary much during the life of the structure, proposals have been advanced with multipliers varying from just over unity to as much as one and one-half. The live load is the indeterminate value. Not only must it represent the first owner's original needs, but it must be adequate to carry construction loads during the building period, it must give the owner whatever latitude may seem desirable to change the use of the building during his ownership, it must give fair value to any successive owner and afford him similar latitude, and it must be at least up to the general standards that the public would expect for the type of building and its location. With so many possibilities for change, it has been suggested that the live load be multiplied by factors varying from as low as 1¾ to as much as 2½. Since the results are in direct linear relationship to the safety factors chosen, and since all of us will freely admit that the relationship of allowable working stress under present codes to the cylinder strength of the concrete is anything but a true measure of the safety factor, considerably more study will have to be given to these values, in spite of the work of the present committee of the American Society of Civil Engineers.

A third point is in the applicability of the method to actual monolithic structures. Test data have mostly been accumulated for single-span, freely-supported, free-standing, individual members. In applying this method to monolithic structures, a number of questions immediately arise, such as, whether the current moment factors, which are invariably obtained by elastic analysis no matter what procedure is used, would equally apply in plastic design and what width of tee or what cross-distribution or plate action to use in distributing loads crosswise of the span.

To elaborate a little, the apportionment between positive and negative moments is invariably obtained by elastic analysis, whether done by coefficients, moment distribution, or other methods. These apply only within the elastic range of the materials. The proposed method develops stresses all the way up to the ultimate strength under multiple loading. This would go well above the elastic range, develop permanent sets, and might result in the creation of semihinges at points of a maximum negative moment where the reinforcing steel yields somewhat but still carries a moment equivalent to its yield

point strength, the remainder of the moment being transferred elsewhere. Against this, it has been pointed out that while we are figuring in the ultimate zone under multiple loadings, we are actually stressing the material only a fraction of this so that we would be in the elastic range, making elastic analysis applicable. There is a certain inconsistency in reasoning to change glibly from the elastic to the plastic range and back again. It is even conceivable that such moment factors as we are now satisfactorily using with uniformly varying stress prisms might not be those which work best with rectangular stress prisms.

Also, the cross-distribution of load in the plastic range might be better or worse than in the elastic one. Beam tees have been arbitrarily established by experience and elastic analysis has been invoked to verify these practical conclusions. It could very well happen that our views should be revised when plastic analysis is employed.

Some other factors that need consideration are those of diagonal tension and bond. Very frequently the design is determined by these items as much as by flexure. The designer would be using multiple loads in determining resistance to flexure. At present, no proposal exists for handling diagonal tension and bond at ultimate values. Certainly the designer does not want to carry two sets of computations, so eventually diagonal tension and bond would be computed on an ultimate basis as soon as data are forthcoming. In applying Whitney's method to several typical buildings, it was found that proportions were very frequently determined by diagonal tension and bond and no better method existed than to handle these under the present methods of the Joint Committee and American Concrete Institute Codes. These very frequently precluded the possibility of taking full advantage of plastic design. Furthermore, it may well be that with the high strength concretes now becoming popular and the higher-bond bars commercially obtainable, the whole subject of web stresses may be open to review and reappraisal and that methods of design can be developed comparable with the plastic method for flexure.

Bond under any conditions is a complicated subject and particularly so under special anchorage. The question arises whether it is necessary to transfer the longitudinal shear from concrete to steel within the same differential length in which it is created or whether it is possible so to distribute the longitudinal shear along the length

of the bar that special embedment, hooking, or other anchorage can absorb a considerable portion of this and relieve the highly stressed portions of the length. This would, of course, affect the moment depth of the beam, causing it to act more like a tied arch, but that may be just what happens. Experiments now under way for the American Iron and Steel Institute will probably shed considerable light on both of these controversial subjects; and if the plastic method for flexure continues to grow in favor, those test results should be analyzed from the ultimate strength angle as well as from the viewpoint of safe working stress.

Plastic sagging or time flow should come in for further study, because the application of plastic design, if carried to the ultimate possibility of balanced reinforcement with rational safety factors, will result in shallower and probably more heavily reinforced members. There have already been instances of appreciable sagging of thin slabs and girders on relatively long spans, due quite apparently to a time-yielding of the concrete. Experiment, investigations, and possibly tests seem indicated to determine how much such plastic yielding can be permitted without seriously affecting the value, use, and occupancy of different types of structures. Until this is done, the designers under the plastic theory will have to use their own best judgment and experience to prevent excessively thin horizontal flexural members. It is probable that the amount of mixing water, brand of cement, and other factors entering into the concrete itself may have as much to do with the plastic yielding of such members as the ratio of depth to span. Additional data on this subject are very desirable before applying the plastic method as the sole means of design.

The relative economy will also have a considerable bearing upon the application of this method. A fairly comprehensive group of studies made for the American Iron and Steel Institute indicated that in ordinary multiple-story, loft buildings, it was usually not practicable to take full advantage of the plastic method for flexure and at the same time meet the present code requirements for share and bond and simultaneously meet preconceived notions of relative depth-to-span ratios. It also became evident that maximum economy frequently did not result with balanced reinforcement. This will vary with the type of structure. The more expensive type of commercial buildings with exterior stone walls, bronze or aluminum sash, and relatively expensive interior partitions is the type of structure where any saving

in total story height considerably affects the cost. Factory and storage buildings with inexpensive walls and no interior partitions, and especially when only a few stories in height, are the type of building where a saving in total story height is not much of a factor; hence, it is impossible to draw any definite conclusions that would apply equally well to all structures. Trying, however, to sum this up as well as possible until better data are available on shear, bond, and plastic deformations, the present designers are more likely to use somewhat deeper sections than balanced reinforcement under the plastic theory would permit and are likely to use the rectangular stress block as a means of varying the amount of reinforcing steel within concrete sections that are only, perhaps, 10 to 20 per cent shallower than those obtainable under the uniformly varying stress method. As long as reinforcing steel is relatively expensive compared with concrete, and even with form work, full advantage of balance reinforcement will not be taken in ordinary structures.

The application of Mr. Whitney's method goes even beyond the region of normal multistory buildings. For one thing, it rationalizes the step-by-step progress from a member totally in compression to one in pure bending. Our present codes design axial-loaded compression members by the plastic method, purely flexural members by the uniformly varying stress prism, and eccentrically-loaded members with a hybrid combination of the two, which, under certain conditions of eccentricity, can be made to produce rather absurd results. Under the plastic method, we would progress through all the different amounts of eccentricity from pure compression to pure bending with no logical inconsistencies and with much better proportioned members.

This, for example, means that the plastic method is particularly applicable in the case of reinforced concrete arches or rigid frames where off-center thrusts are universally encountered.

The plastic method also offers unusual opportunities in longer spans, especially flat slabs, but probably also other types of construction. Parking garages, certain types of manufacturing plants, and even office buildings have found that column spacings in the neighborhood of 50 to 55 feet offer considerable advantages in the use of the floor space. Even insurance offices report that a 54-foot column spacing is probably the most satisfactory for batteries of desks. Here, the savings by the plastic method double up rapidly as any decrease in concrete outline is immediately reflected as a saving in dead weight,

which still further saves concrete. It might almost be said that certain structures can only be satisfactorily analyzed and their continued good service explained by recourse to this method.

The general purpose of the above random remarks is to point out that while in general accord with this more scientific approach to the analysis of reinforced concrete structures, it is felt that considerably more light needs to be shed upon such matters as diagonal tension and bond and that more knowledge is necessary as to whether certain current methods of elastic frame analysis are applicable, also how much plastic yielding is permissible in various types of structure, before the method can come to general use and be recognized by the codes as on a par with the uniformly varying stress prisms, which undoubtedly all of us recognize has but little correlation with the real facts and is merely a computer's aid in obtaining results which experience has demonstrated are workable.

Discussion by Dean Peabody, Jr., Member*

This important paper on a Plastic Theory of Reinforced Concrete Design will probably be reviewed by engineers who have used it or contemplate its use. Therefore, I shall confine my discussion to a consideration of the effect of the substitution of the plastic theory for the present working-load theory upon the teacher and student of reinforced concrete design.

For nearly twenty years the straight-line working-stress theory has been in the process of disintegration. The Joint Committee and A.C.I. building codes have added a patch here and there to a formerly logical, homogeneous theory of stresses produced by loads only. The instructor becomes apologetic whenever he discusses members containing steel in compression. The morale of the student suffers after he has laboriously computed for beams the stresses in the compression steel and then multiplies this result by a correction factor of 100 per cent. The column theory, at present, is in the extremely illogical situation where one uses a plastic theory for axial loads and the straight-line theory for columns with eccentric loads. I defy any instructor to avoid a feeling of inadequacy as he endeavors to give our present column theories a harmonious presentation.

The instructor, or student, adopting the Whitney plastic theory must accept the assumption that a rectangular stress block of depth a and intensity $0.85 f_c'$ gives the same resultant compression force on the same line of action as the varying stresses at failure shown in Figure 1. He must again accept the special constants for simultaneous failure of concrete and tension steel for balanced design (Equation 2). The only alternative is to laboriously determine these relations for himself by analysis of the available test data. We must not forget, however, that the use of the A.C.I. Code (for instance) is an act of faith in those who have established the allowable stresses and methods of procedure. Long time experience will justify or discredit such faith. In the past we have disregarded shrinkage and plastic flow stresses, knowing that they exist, and have compensated by using comparatively low allowable stresses due to loads. The tests upon which the plastic theory is based include the effect of loads, shrinkage, and considerable plastic flow. A suitable factor of safety should give safe working conditions. It is rather remarkable that the same factor of safety applied to beams and to columns with large eccentricity of

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loading will give section dimensions and steel areas in close agreement with present designs. This is also true for columns with small eccentricity of loading, if one uses the 80 per cent reduction of load for tied columns that the present codes specify.

Why bother to change if comparable designs can be made by the two theories? From the instructional point of view the empirical plastic theory equations are superior because they can be presented by simple, harmonious derivations. Also, Figure 5 shows that the use of 2 to 2.5 per cent of steel enables one to use a much smaller section by the plastic theory to withstand the same bending moment. This may well eliminate compression steel at the supports of tee beams. The author has also discussed the unexploited area in Figure 6 between straight-line and plastic theories for eccentrically loaded columns.

Figures 4, 7, *et seq.*, make it evident that the "short cut" tables and plots of the plastic theory are as easy to manipulate as those we now use. Designs may well take less time as soon as the designer is familiar with the new procedures, while there will be a wider range of section dimensions and steel percentages from which to select the economical sizes.

As yet no recommendations are made for bond, shear and diagonal tension computations for members stressed to failure. The author rightly observes that these also merit an authoritative discussion before a plastic theory is complete.

However, it is probable, as Mr. Whitney suggests, that a plastic theory would be introduced by permitting either the old or new theories to be used. Imagine then the predicament of the instructor who must present two theories to the student already bewildered by the multiple details of any reinforced concrete design!

Discussion by Oliver G. Julian, Member*

Mr. Whitney is to be commended for presenting to the profession a method of strength analysis which is at once realistic, practical and simple. It is hoped that his method is generally adopted in the near future. The subject paper should be read against the background of the following listed publications; these are in addition to the references cited by the author.

a) "Theory of Limit Design" by J. A. Van den Broek and discussions, Transactions A.S.C.E., Vol. 105 (1940), p. 638. For comments regarding advantages of strength as compared to stress analyses.

b) "The Mathematical Theory of Elasticity" by A. E. H. Love, Chapter IV, "The Relation between the Mathematical Theory of Elasticity and Technical Mechanics", Fourth Edition, Dover Publications (1944), p. 112. For discussion regarding the limitations of the theory of elasticity.

c) "Effect of Time Element in Loading Concrete" by W. K. Hatt, Proceedings, A.S.T.M., 1907, p. 421. Professor Hatt was among the earlier investigatiors of time effect.

d) "Plastic Flow in Concrete Arches" by Lorenz G. Straub, and discussions, Transactions, A.S.C.E., Vol. 95 (1931), p. 613.

e) "Deformation of Steel Reinforcement During and After Construction" by Sergius I. Sergev, Transactions, A.S.C.E., Vol. 99 (1934), p. 1343.

f) "The Plastic Flow of Concrete" by J. R. Shank, Ohio State University, Engineering Experiment Station Bulletin No. 91 (1935).

g) "Concrete Under Sustained Working Loads; Evidence That Shrinkage Dominates Time Yield" by G. A. Maney, and discussions, Proceedings A.S.T.M., Vol. 41 (1941), p. 1021.

h) "Ultimate Design of Reinforced Concrete", published by The Portland Cement Association as Bulletin No. 11 (1946).

i) "Tests Verify Accuracy of Transformed Area Method of Predicting Beam Stresses" by J. Trueman Thompson, Civil Engineering, July 1947 and discussion published under heading, "Transformed—Area Method of Concrete Analysis Questioned" by George C. Ernst, Civil Engineering, Sept. 1947, p. 44.

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j) "Plastic Flow of Thin Reinforced Concrete Slabs" by George W. Washa, Journal A.C.I., November 1947, p. 237.

As has been noted by numerous investigators over a period of four decades, concrete is at once a visco-elastic[1] and a brittle material. When subjected to loads of protracted duration, hardened concrete like rock, clay, green timber, steel at high temperatures, lead and sealing wax acts like a viscous fluid; it flows continuously with time, at least for a number of years. On the other hand, when the time-rate of loading is rapid, as for impact or high frequency cyclic forces (of the order of 60 cycles per sec.), such as those to which the supports of high speed machines and some bridges are subjected, it may behave as an elastic material, like glass with constant elastic modulus almost up to the point of failure;[2] in this case the concrete has not time to flow viscously and the values of both the elastic modulus and strength may be considerably greater than those pertaining to similar members subjected to loads of protracted duration or to ordinary test procedure. It appears obvious that an attempt to define the elastic modulus for such a material as concrete without recourse to parameters representing at least the duration and velocity of loading, must be futile; the errors may be of the order of several hundred per cent. When concrete fails, the total change in length along trajectories of stress is small and the failure is sudden. For example, the total change in length of a specimen of 6,000 p.s.i. concrete, tested in compression, just prior to failure may be of the order of .2%; the corresponding figure for 2,000 p.s.i. concrete may be of the order of .4%. Concrete is, therefore, generally recognized as a brittle material.

What has been said regarding concrete *per se* may also be said, to a lesser degree, regarding reinforced concrete; for loadings of protracted duration or which are applied slowly the concrete flows viscously, transfers stress to the reinforcing steel until the latter is stressed up to the yield point, after which the concrete and steel share the stresses necessary to maintain equilibrium approximately in proportion to their ultimate strength and yield point respectively.[3] This is the key note of Mr. Whitney's method. Its validity for loadings of protracted duration or which are applied slowly seems reason-

[1]For definition see "The Influence of Time Upon Creep" by A. Nadai, "Stephen Timoshenko 60th Anniversary Volume", published by The Macmillan Co., p. 155.

[2]See "The Safety of Structures" by Alfred M. Freudenthal, Proceedings A.S.C.E., Feb. 1947, Fig. 10 and adjoining text, p. 209.

[3]The yield point and ultimate strength of concrete may be said to be identical or the yield point of concrete may be said to be non-existent.

able *a priori* and has been amply proven by numerous independent tests. So far as the writer knows, the theory has not been proven for impact or high frequency cyclic loads where the time-rate of applying stress may be of the order of 50,000 (or more) p.s.i. per second (as compared to a standard testing rate of 25 to 50 p.s.i. per second). Its application to such cases seems somewhat doubtful. This does not detract materially from its value as applied to ordinary buildings and many bridges. For ordinary buildings, the effect of impact and cyclic loads, although important, are seldom governing; to a lesser degree this applies to many bridges. When the effect of impact and high frequency cyclic forces must be taken into consideration, the classical method of transformed sections with appropriate values of E_c can be used. Such values of E_c should be obtained from tests of concrete similar to and conducted at the time-rate of loading that will prevail in the prototype. On account of the many uncertainties inherent in E_c, and the velocity of loading, it will ordinarily be found advisable to use two limiting values of these parameters and the corresponding transformed section; one set of which surely will be high, whereas the other set surely will be low. The true action of the structure will then be reflected by some value intermediate between these limits. It should be emphasized that the elaborate character of the procedures usual in the classical method conceals manifold arbitrary assumptions, such as linear stress distribution, the value of E_c, the extent and the effect of tension cracks, the effect of neglecting shrinkage, etc., on which the procedures are based and may inspire an unwarranted confidence in the reliability of the results. As stated by Professor Hardy Cross, "The standard method of stress analysis using the transformed section has little rational and no experimental basis."[4] This is especially true in case the working stresses are of the order of those presently given in standard codes; e.g. $f_c = .45\, f_c'$ for gravity loads only and $.60\, f_c'$ with the effect of wind included.

The compression elastic modulus of concrete depends upon, at least, the following listed conditions: (1) magnitude of the stresses, (2) time-rate of applying load, (3) duration of load, (4) age of concrete at time the load is applied, (5) moisture content of the hardened concrete up to and at the time the load is applied, (6) curing conditions including ambient temperature and humidity prevailing since

[4] "Design of Reinforced Concrete Columns Subject to Flexure", Proceedings, A.C.I., Vol. 26 (1929-1930), p. 168.

the concrete was placed, (7) water-cement ratio, (8) air entrained in the concrete, (9) gradation of the aggregate, (10) shape of the aggregate particles, (11) quality of the aggregates, their elastic moduli, plasticity ratios, etc., (12) quality of the cement, (13) concrete proportions, (14) mixing and placing conditions, (15) the stress conditions including release and reapplication of prior loads prevailing throughout the history of the concrete. Numerous investigators have attempted to define E_c for various stress conditions in terms of the ultimate strength of test specimens. The reported results differ widely, a few have been summarized elsewhere.[5] For a given concrete (testing say 5,000 p.s.i.) having a given history typical values of equivalent E_c and n in the usual working stress range, including the effect of shrinkage and viscous flow might possibly be as listed below.

Loading Condition	E_c p.s.i.	n
Protracted loading such as dead load	1×10^6	30
Short time loading such as live load on a bridge	5.5×10^6	5.5
High speed cyclic loading	7×10^6	4.3

Within the range of usual working stresses, E_s is substantially constant and equal to approximately 30×10^6 p.s.i. However, as ultimate static loads are approached E_c and E_s have little or no practical significance, (at such loads the ratio of non-elastic to elastic deformation is so great that the latter may be neglected). Also at such loads the effects of temperature changes, shrinkage and viscous flow over a protracted period became insignificant. By freeing us of the practically impossible task of evaluating E_c and these effects, Mr. Whitney's method greatly simplifies reinforced concrete design and should be considered as a boon to the profession.

Being a strength theory rather than a stress theory, the subject method places emphasis on the ratio of the loads which may cause failure to the loads the structure is calculated to resist in service and thereby allows us to view this ratio as a realistic factor of safety, or factor of overload. It should be (but unfortunately is not) generally recognized that the ratios of the strength of the steel to the working stress in the steel and the strength of the concrete to the working stress in the concrete, as computed by classical methods, are not true

[5]"Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete" by the Joint Committee, discussion by the writer, Proceedings, A.S.C.E., February 1941, Fig. 1, p. 252. See also "Relationship Between Strength and Elasticity of Concrete in Tension and in Compression." Engineering Experiment Station, Iowa State College Bulletin 90 (1928).

measures of the safety of a structure and bear no consistent relation to the factor of overload. If we multiply the loads which tend to destroy a structure by factors of overload and then, using stresses which approach the ultimate, design the structure to resist these increased loads (together with the other loads on the structure) we have a much more realistic view of the safety of the structure. The factors of overload may contain factors of uncertainty which are measures of the degree of accuracy with which the maximum probable loading can be calculated practically. The point of this paragraph is well illustrated by Mr. Whitney's figures 6 to 9 inclusive pertaining to axial load combined with moment.

Mr. Whitney states that, "it will be necessary to revise thoroughly the methods of design for bond and shear stresses". The writer is thoroughly in accord with this statement. His thoughts which follow on these most important but controversial questions are not made with the idea of being final but rather with the thought of promoting discussion.

As stated by Professor Mylrea,[6] "Just as longitudinal reinforcement is provided where longitudinal tension occurs in a reinforced concrete beam so, in the web, reinforcement should be provided to resist diagonal tension. The two phenomena are in no wise different from each other, the curve of tension being continuous along a stress trajectory." Furthermore, as stated by the late Professor George F. Swain,[7] "It is obvious that if the beam is to be designed on the assumption that there can be no tension whatever in the concrete, there must be diagonal tension bars at every section where there is shear, and the sum of the vertical components of the stresses in these bars cutting any vertical section must equal the shear on the section." These statements, being based on pure statics, appears to be just as true for beams stressed to the ultimate as they are for members stressed within the range of working stresses. For the case of balanced single reinforcement, at sections where the moment is close to the ultimate, the distance c between the centroids of longitudinal compression and tension will approximate $\frac{3}{4}\,d$. In case the reinforcement is less than that required for balanced design or if compression reinforcement is used c will be somewhat more than this amount. Also, at sections where the moment is small the concrete stress block will

[6] "Studies of Shear in Reinforced Concrete Beams" by T. D. Mylrea, Transactions, A.S.C.E., Vol. 94 (1930), p. 735.

[7] "Strength of Materials" published by McGraw-Hill Co., 1924, p. 375.

be nearly triangular and c or jd will probably be greater than $\frac{3}{4}\,d$; it will usually be about $\frac{7}{8}\,d$. In some cases the concrete alone may be adequate to resist the diagonal tension. (This also applies to longitudinal fiber stress in unreinforced concrete beams provided the member is not so restrained that it will be cracked on the tension side due to shrinkage and temperature effects only.) Tests by Gonnerman and Shuman[8] indicate that the unit tensile strength f_t of concrete varies from about .11 f_c' for low strength concrete ($f_c' = 2{,}000$ p.s.i.) to less than .07 f_c' for high strength concrete ($f_c' = 9{,}000$ p.s.i.). It is probably safe to consider that the unit tensile strength is represented by

$$f_t^2 = 25\, f_c'. \tag{a}$$

Based on this value and the value of c being $\frac{3}{4}\,d$, the following formulae are tentatively proposed for members of constant depth.

For beams which are not reinforced for diagonal tension

$$V \leqslant \frac{15}{4}\, db \sqrt{f_c'} \quad . \tag{b}$$

In case the shear at a section exceeds the value given by equation (b), it appears to the writer that the member should be fully reinforced to resist all the diagonal tension incident to shear, the concrete being considered as ruptured. If there are several sets of web bars, the tension in a single set in a plane may then be approximated as

$$T_v = \frac{4}{3}\,\frac{Vs}{(\cos\alpha + \sin\alpha)\, d}\,, \tag{c}$$

in which α is the angle the set of bars make with the longitudinal axis of the member and s is the distance between sets of bars, measured along the axis of the member. Equation (c) is not intended to apply for α less than 15 degrees. If there is but one set of bars resisting the shear, the tension in that set approximates

$$T_v = \frac{V}{\sin\alpha}\,. \tag{d}$$

In equations (c) and (d), T_v should not exceed the cross sectional area of the web steel, at the section, times the yield point. These equations will be recognized as having the same form as those given

[8] "Compressive, Flexure and Tension Tests of Plain Concrete" by H. F. Gonnerman and E. C. Shuman, Proceedings, A.S.T.M., Vol. 28 (1928), p. 527.

in "Building Regulations for Reinforced Concrete" of the A.C.I. (1941 edition). However, it should be noted that V is the total shear at the section, rather than the "excess of total shear over that permitted on the concrete". When employing the working stress concept, it has been customary to assume that there is a reduction in T_v due to the concrete above the cracked region resisting vertical shear. Whereas, this practice may possibly be permissible when considering stresses below the range where viscous flow is appreciable, it is difficult to justify for higher stresses. In this connection, Professor Mylrea stated,[9] "In the day when high shear values will be allowed this reduction will not be of great consequence, and it will be better to neglect it entirely. In fact, it will then be quite possible that owing to the gradual time yield in shear of the concrete above the crack, the full shear V, must be taken by the web reinforcement, and it will be more nearly divided equally among the bars crossing the crack."

In addition to adhesion, the ultimate bond strength of concrete depends upon the shearing, crushing and tensile strength of the material, and upon time yield. It is also affected very largely by the position of the bars and the pains taken to insure that the concrete is in intimate contact with all parts thereof. Bars cast in the top horizontal position should receive special consideration; for such bars the bond strengths indicated in the following may be somewhat daring. Contrary to past prevalent belief, it is now generally recognized that bond strength is not a linear function of the cylinder strength of the concrete in compression and the length of embedment. Even assuming good workmanship, it is questionable if we can depend upon the unit bond strength per square inch exceeding the value represented by

$$u^2 = 36\, f_c'. \tag{e}$$

Assuming that the stresses in various bars of a group are equal, the bond force per unit of length V/c is distributed among the bars in proportion to their areas. Then taking c or jd as $\frac{3}{4}\, d$,

$$u_1 = \frac{4}{3}\,\frac{V}{d}\,\frac{D_1}{\pi \Sigma D^2} = \frac{4\, V\, D_1}{3\, d\, \Sigma_0 D}, \tag{f}$$

in which u_1 = the bond stress for a bar having a diameter D_1, ΣD^2 = the sum of the squares of the bar diameters, and $\Sigma_0 D$ = the sum of the products of the bar perimeters by their diameters. Provided the bars are all the same size, equation (f) reduces to

[9] Loc. cit., p. 743.

$$u = \frac{4\,V}{3\,d\,\Sigma_0}. \qquad \text{(g)}$$

The bond stress should not exceed the value given by equation (e). Hence according to the above, the shear V should be not greater than

$$V = 14.1\,\frac{d\,\Sigma\,D^2}{D_1}\sqrt{f_c'}\,, \qquad \text{(h)}$$

in which D_1 is the diameter of the largest bar in the group. For bars all of the same size, equation (h) reduces to

$$V \leqslant 14.1\,d\sqrt{f_c'}\,\Sigma\,\mathrm{D} = 4.5\,d\sqrt{f_c'}\,\Sigma_0. \qquad \text{(i)}$$

For test data regarding bond, reference may be made to "Bond Between Concrete and Steel" by H. J. Gilkey, S. J. Chamberlin and R. W. Beal, Bulletin 147, Iowa Engineering Experiment Station (1940), "Comparative Bond Efficiency of Deformed Concrete Reinforcing Bars" by Arthur P. Clark, Journal of Research of National Bureau of Standards, Vol. 37, No. 6 (December 1946), p. 399, and "Bond Characteristics of Commercial and Prepared Reinforcing Bars" by S. T. Collier, Journal, A.C.I., June 1947, p. 1125. The first of these references contains an extensive survey of the literature prior to 1940. It may be that the use of bars with high, closely spaced, transverse ribs will warrant the use of somewhat higher ultimate bond stresses than those indicated by the formulae given above. It should be noted that bond slip has the same effect as stretch of the reinforcing steel and that these effects are additive.

Mr. Whitney does not mention the important case of axial load combined with biaxial flexure to which practically all columns are subjected. This case might well be made the subject of a separate paper. It is believed, that it may be handled advantageously by means of the ellipse of inertia in a manner somewhat similar to that suggested by Jaroslav J. Polivka in his discussion of a previous paper by Mr. Whitney.[10]

The subject paper prompts the question: If the members in a frame are to be designed by methods reflecting their ultimate strength, should not the frame be proportioned by the "Theory of Limit

[10]"Plastic Theory of Reinforced Concrete Design" with discussions, Transactions A.S.C.E., Vol. 107 (1942), p. 307.

Design" proposed by Professor Van den Broek[11] or other similar methods?

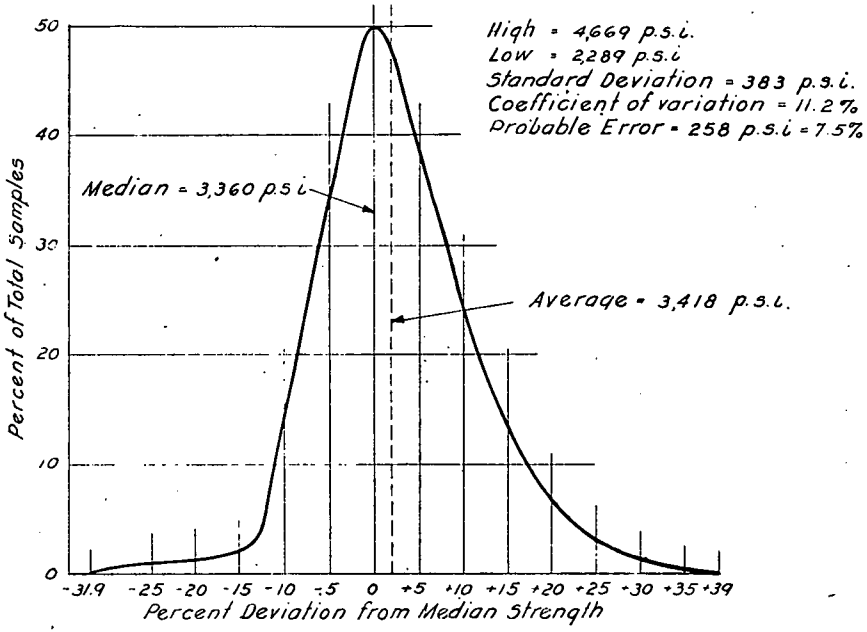
The writer cannot fully agree with Mr. Whitney's statement which reads:

> "Skill in that field [concrete construction] is now so highly developed that engineers may use the material with confidence any place in the world where it is economically practical."

There is no doubt that the statement quoted above should be factual. The profession knows how to make good realiable concrete. However, one only has to make a casual inspection of many recently built concrete structures to realize that this "know how" is not generally practiced. This may be due largely to the all too prevalent notion that concrete need not be manufactured, placed and cured under strict supervision of trained personnel. It is all too often manufactured, installed and *not* cured by those who have not read the specifications, would not understand or be sympathetic with them if they did and who claim to be "concrete experts" simply because they have been installing a product containing Portland cement for the past twenty years or so without having viewed its subsequent condition and studied the reasons for its many failures. Concrete technology, specifications, materials and plant have been improved immensely since the first Joint Committee report of 1909 but general practice has only succeeded in adding to the number of eyesores, and structures requiring inordinate maintenance or replacement.

For work which is conducted under strict supervision, the writer's observations roughly agree with the results given in Mr. Whitney's Table 3. However, for work which is not supervised strictly, the coefficients of variation may be several times those there shown. Figures A and B accompanying this discussion show typical frequency curves for concrete strengths where the product was manufactured under excellent supervision. These curves are for concrete installed under two widely different sets of conditions, for different purposes and in different parts of the globe. The writer heartily agrees with Mr. Whitney's statement reading, "A well managed job should not be penalized because many jobs are poorly controlled. If the control is so poor that the proper margin of safety in the mix design cannot be estimated, concrete should not be used as a structural material." This may be interpreted as—concrete should not be manufactured or

[11]Transactions A.S.C.E., Vol. 105 (1940), p. 638.

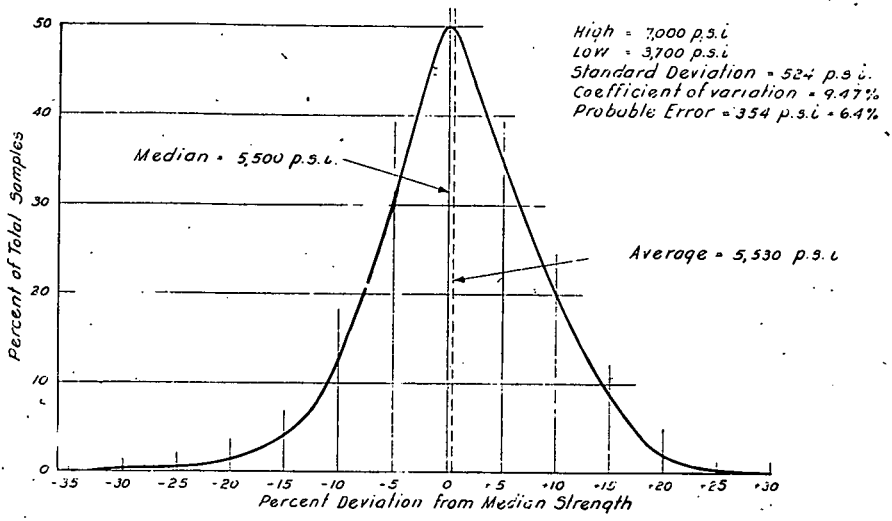


RESULTS OF 28 DAY TESTS
OF 564 CYLINDERS REPRESENTING
85,000 CU. YDS. OF MASS CONCRETE

FIG. A

installed except under strict supervision and after all the ingredients have been tested and found to be satisfactory. Even under these conditions, the writer is inclined to believe it is well to aim the median concrete strength 20% above the figure used for f_c'. This to allow for variations in the product for which numerous excuses will be found; and with strict supervision will probably result in all but a minor proportion of the concrete testing above the value used for f_c' in design. For additional data on the variations which may be expected, reference may be made to "Manufacturing Concrete of Uniform Quality" by William M. Hall and discussions[12] and "Ordinary Concrete" by Miles N. Clair.[13]

[12]Transactions A.S.C.E., Vol. 96 (1932), p. 1336.
[13]Proceedings A.C.I., Vol 35 (1938-1939), p 257.



RESULTS OF 28 DAY TESTS
OF 6.30 CYLINDERS REPRESENTING
18,200 CU YDS. OF BUILDING CONCRETE

FIG. B

It will be noted that for the reinforcing steel, tests of which are cited by Mr. Whitney, all specimens tested showed yield points exceeding the specified minimum by 10 to 46%; a total spread of but 36%. For frequency curves indicating the results of tests on the silicon steel used in the Golden Gate Bridge, reference may be made to "The Safety of Structures" by Alfred M. Freudenthal.[14] With full cooperation of all parties concerned, it should be practical to obtain concrete which will consistently test within such limits as those indicated for these two cases of steel.

[14]Proceedings A.S.C.E., October 1945, p. 1166.

Closing Discussion by Charles S. Whitney

The writer wishes to thank Prof. Peabody, Mr. Reese and Mr. Julian for their discussions which have made a very valuable contribution to the subject. He will resist the temptation to discuss the many interesting and important points they have raised and limit the closing discussion to a few remarks.

Mention should be made of extensive tests on the effect of repeated loading reported by M. B. Le Campus[18] and by Prof. Dr. Ing. h.c. M. Ros.[15] The tests reported by Le Campus were made on various types of plain and reinforced concrete specimens under loads repeated at the rate of 500 times per minute. The limit of fatigue strength was compared with the strength under static load with the following results:

Ratio of Limit of Fatigue Strength to Static Strength

Plain concrete, compression	0.62
Plain concrete, bending	0.59
Bond	0.69
Reinforced concrete, flexure, tension failure	0.54
Reinforced concrete, flexure, compression failure	0.60
Shear, inclined stirrups	0.41
Shear, vertical stirrups	0.35
Shear, no stirrups	0.42

The report recommends that members subjected to repeated loading could be designed in the same way as members under static load if the variable portion of the load is multiplied by two for flexure or three for shear.

The report by Ros indicates that the fatigue strength of reinforced concrete beams under flexure shows a good correlation with the equation,

$$M_f = pf_{sf}bd^2\left(1 - \frac{pf_{sf}}{2f_{cf}}\right)$$

This is the same as Eq. (1) with f_s replaced with the fatigue strength of the steel (f_{sf}) and $0.85\,f'_c$ replaced with f_{cf}, the fatigue strength of concrete in compression. These fatigue strengths will vary with the range of loading and the number of repetitions and therefore this equation provides a method of estimating the fatigue strength of a beam under different conditions of repeated loading.

Both of these reports indicate that the plastic theory can be used as well for cases of dynamic loading as for static loading.

The adoption of the plastic theory and the method of ultimate strength design will require a rather thorough revision of the designer's thinking in order to avoid confusion. To cite one example, the term "balanced reinforcement" means the amount of steel theoretically required to develop the full allowable compression strength of a concrete beam in flexure. In this case both the steel and the concrete theoretical unit stresses reach the maximum allowable intensity under the same bending moment and the term seems to carry the implication that this is a desirable condition.

Balanced reinforcement may often be desirable when the straight line method is used because of the very low limitation it places on the steel ratio; but that is not generally true with the plastic theory unless the term is used in a broader sense. In general, it will not be best to use the full steel area needed to develop the maximum flexural strength of the concrete in compression because of a number of other considerations mentioned by Mr. Reese.

The flexural strength of concrete is only one of a number of important factors which should control the design. If "balanced reinforcement" is to be the goal in beam design, it must mean the amount of steel which will provide the best balance between the steel and concrete strength, bond strength, shearing strength, cost of construction, deflections, available space for reinforcement and any other limitations existing in the particular structure.

The same is true of course of the straight line theory but to a lesser extent because the unreasonably low steel ratio limitation places such restraint on the size of the member that the other factors may not be so critical. The partial release of these arbitrary restrictions by the use of the plastic theory leads Mr. Reese to caution that the plastic theory should not be used until further information is available regarding shear, bond, deflections, etc.

The writer has stated that full advantage of the plastic theory cannot be taken until there has been further research, but he believes that it would be much better to adopt it now with the present limitations on shear, bond and deflections. There appears to be no reason for waiting for years for more information on those factors which can actually be more or less isolated from the problem of flexural design covered by the plastic theory equations.

OF GENERAL INTEREST

THE MAINE TURNPIKE

By Ernest L. Spencer

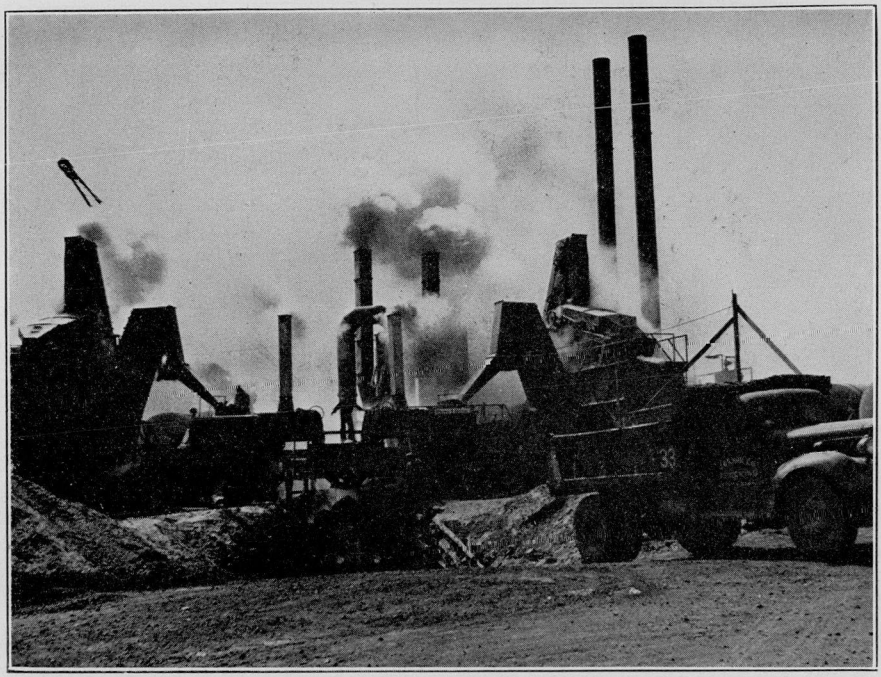
On Saturday, September 27, some 70 members and guests of the Boston Society of Civil Engineers left Park Square, Boston, aboard two buses bound for an inspection trip of the first 44 mile section of the Maine Turnpike. The party met its guide, Mr. L. D. Brown, project engineer for the consulting engineers Howard, Needles, Tammen & Bergendoff, who are responsible for the designs, plans, and supervision of construction for the Turnpike Authority, on the toll plaza of the Maine-New Hampshire Interstate Bridge between Portsmouth, New Hampshire and Kittery, Maine.

The group proceeded along Route One to York Center and then inland a short distance to the Turnpike. The first point of interest was the inspection of the 610′ multiple span bridge across the York River. This bridge is a continuous beam span type of bridge resting on pile bents. The piles, 14″ WF 89# sections and encased in the tidal range by 30″ steel pipe, were driven by a #1′ Vulcan hammer to a 100 ton bearing as computed by the Engineering News formula 85′ through clay to shale. The casings were cleaned out and filled with concrete and a concrete cap extending from 2′ below the water level up to the beam spans. About 8′ clearance was provided for boats. The beam spans were continuous over 2 supports and hangers provided at the ¼ points. These hangers acted as expansion joints as well.

As the party continued northward, it was explained that the roadway alignment was designed according to the latest design standards of the American Association of State Highway Officials. The maximum curvature without superelevation is 1°. The non-passing sight distance was set at 525′ from a height of 4′-6″ to an object 0′-4″ above roadway. The steepest grade is 5% and this is only for a short distance.

It was noted that there were 4-12′ lanes, two in each direction separated by an 18′ elevated grass median strip. This median strip has a 4′ paved shoulder on each side. Along the outer edges of each roadway there was an 8′ paved shoulder for parking purposes. These shoulders were constructed of compacted gravel and given an asphaltic surface treatment. It was explained by Mr. Brown that these would permit fullest use of the travelled way by moving vehicles and make snow removal easier as well as help reduce icing.

It was interesting to note that this median strip was maintained throughout the entire job. This required two separate bridges for each two lane roadway over both the Saco and York Rivers. It also made the numerous traffic overpasses on the Turnpike require larger spans. Furthermore, Mr. Brown pointed out that this elevated strip sloped from its center line to the outer edge where a series of roadway



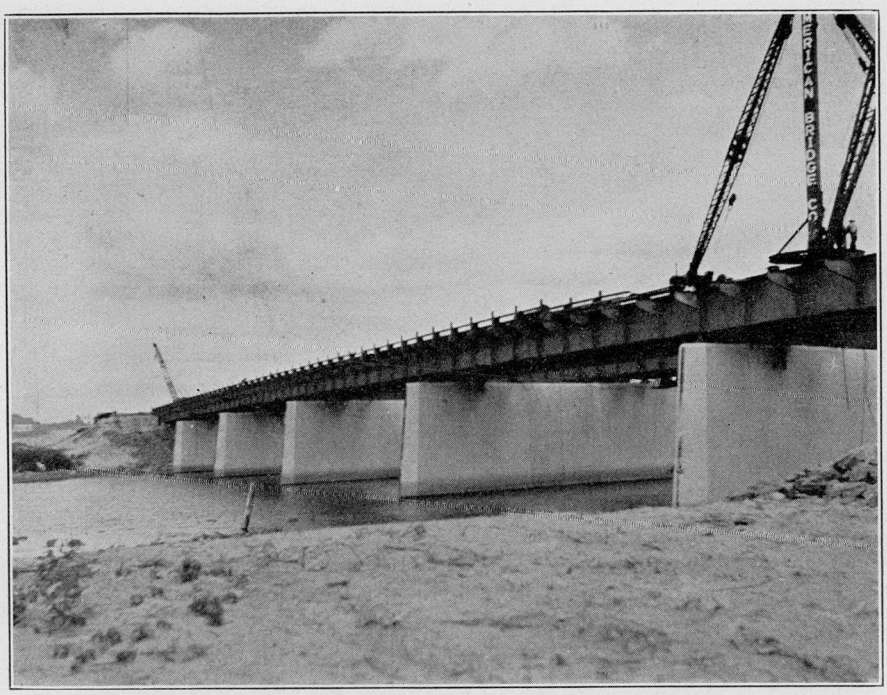
ASPHALTIC CONCRETE PLANT

drains were provided. This was felt to be superior to the depressed median strip with the drains along the centerline in preventing icing and facilitating snow removal.

The next stop was to watch some of the Barber-Greene paving machines at work. The roadway proper was being layed on about 2′ to 4′, depending upon conditions, of coarse gravel fill selected so as to reduce any frost heave to a minimum. This gravel was considered satisfactory if not more than 10% of the fines passed a #200 mesh sieve. Fill material was compacted to 95% of maximum density at optimum moisture content as determined by tests in the laboratory.

The Turnpike pavement is a two course machine spread asphaltic concrete. The base course, 6″ thick and spread in 2-3″ layers, was a hot mix using local gravel. The maximum percentage of wear by the Los Angeles Abrasion Test was limited to 40% and it was graded from 1½″ down to dust. The wearing surface was a 2″ layer of hot-mix using a crushed granitic stone. The maximum percentage of wear permitted was 20% (actually, the maximum was 17%). This stone was graded from ¾″ down to dust. Both courses required an asphalt cement of 85-100 penetration and the base course required from 4-6% whereas the wearing surface required 5-8% by weight. This type of pavement, according to the consulting engineers, was selected because of "adaptability to snow and ice conditions, economy, freedom from roadway joints and other reasons of local import."

A visit to the asphaltic concrete mixing plant followed. This plant was adja-



SACO RIVER BRIDGE

cent to the roadway near Wells. It consisted of two portable Barber-Greene continuous type mixer plants. Operation continued 24 hours a day. The two plants discharged simultaneously into the hauling trucks on a suppressed ramp. All aggregate was dried and heated using rotary dryers, and graded by rotating sieves before being introduced to the pugmill for mixing with the asphalt.

After an enjoyable luncheon at the Kennebunkport Inn, the party continued on up the Turnpike. Inspections were made of the traffic interchanges at Biddeford and Saco. The Wells interchange was viewed before lunch. These interchanges provide 1,000 acceleration and deceleration lanes for traffic entering or leaving the Turnpike.

The mid-point of the Turnpike provides for a toll house to collect the fees from those using the highway. This area will also have a restaurant and rest room facilities. Those motorists using only a part of the 44 mile length will pay proportionally, it was explained. Identification tickets will be picked up at the point of entrance—there being only three intermediate and the two terminal means of entering or leaving the roadway.

The next point of interest was the Saco River bridge or bridges. This consisted of a six span plate girder deck construction with a total span of 620′. The contractor on this bridge as well as the York River Bridge was Ellis C. Snodgras.

The deck of this structure as well as that over the York River was to be made with a Type II-A cement from Thomaston, Maine. The air entraining agent was Vinsol Resin and the air produced was 4-5%.

The party continued on its way to Portland over the Turnpike before leaving for Boston. The return trip was made over the Turnpike as far as Saco and the remainder of the way over route 1.

The entire roadway is being constructed on a 300 ft. right-of-way purchased by the Maine Turnpike Authority as created by act of the State Legislature. This 44 mile stretch is the first of the projected 400 mile route from New Hampshire to Canada. In this first short link, there are 43 bridges and grade separation structures. Most of the latter consists of deck type, two span continuous beam bridge. The beams were welded for continuity at the centerline support. The Lane Construction Company were the contractors on about 22 miles of grading and all the overpass structures. The Savin Construction Corporation were the contractors on the remaining 22 miles of grading. B. Perini and Sons were doing all of the paving.

The entire 44 mile stretch required clearing 730 acres, 6,000,000 cu. yds. of common excavation, 260,000 cu. yds. of rock excavation, 160,000 feet of piling, 32,000 cu. yds. concrete, 4,000 tons of structural steel and 1,380,000 sq. yds. of asphaltic concrete paving.

The cost of the project is estimated at $15,000,000 all of which was raised by sale of revenue bonds. These will be repaid by the tolls collected from the users of the roadway.

Grading operations began in May, 1946, and the roadway is expected to be open to traffic in November, 1947.

THE WILLIAM THOMPSON SEDGWICK LABORATORIES OF SANITARY SCIENCE AT MASSACHUSETTS INSTITUTE OF TECHNOLOGY

On Wednesday, December 3, 1947, the William Thompson Sedgwick Laboratories of Sanitary Science, a memorial to Professor William Thompson Sedgwick (1855-1921), were dedicated and inspected by a group of about 150 men and women interested in the development of education in Sanitary Engineering. In the belief that this dedication program and a brief description of the laboratory facilities would be of interest to a large number of the readers of this journal, the following has been published:

THE EDITOR

DEDICATION PROGRAM

I. *Exercises 4:00-5:00 P.M.*

a. Introductions
   Dr. John B. Wilbur, Head, Department of Civil and Sanitary Engineering

b. Professor Sedgwick—His Life and His Work
   Dr. Samuel C. Prescott, Professor of Industrial Biology, Emeritus

c. Laboratory Training and Sanitary Engineering Practice
   Mr. Arthur D. Weston, Chief Engineer, Mass. Department of Public Health

d. The Laboratory and Sanitary Engineering Education
   Dean Gordon M. Fair (XI '16), Dean of Engineering, Harvard Graduate School of Engineering

e. The Laboratories and Laboratory Directors
   Professor Wm. E. Stanley, Professor of Sanitary Engineering

II. *Inspection of Laboratories 5:00-6:00 P.M.*

Introduction

By Dr. John B. Wilbur

On behalf of the Department of Civil and Sanitary Engineering I would like to express our pleasure in welcoming you this afternoon to participate in the dedication exercises for the William Thompson Sedgwick Laboratories of Sanitary Science. It is gratifying that you can be with us on this happy occasion.

Preliminary planning for the new laboratories began in the summer of 1944, but it was not until April of 1945 that the Corporation of the Institute authorized the use of a portion of the Institute's Sedgwick Fund for the proposed expansion of our educational and research facilities in Sanitary Engineering.

During the two and a half years that followed, the planning, construction and equipping of these laboratories have progressed to a point where they are now essentially completed. As we dedicate these laboratories we are aware of our debt to the Administrative officers of the Institute, to the members of the Corporation Visiting Committee for the Department, and to the staff of our Sanitary Division. Their joint efforts have made the new laboratory possible.

In dedicating these laboratories today, it is fitting that we pay tribute to the man for whom they are named. We are fortunate in having with us a former Dean of Science at M.I.T., who was associated with Professor Sedgwick as a member of the Institute staff from 1894 until 1921, when he succeeded Professor Sedgwick as head of the Institute's Department of Biology and Public Health. It is a pleasure, as well as a privilege for me to present Dr. Samuel C. Prescott, who will speak on "Professor Sedgwick—His Life and His Work".

Professor Sedgwick—His Life and His Work

By Dr. S. C. Prescott

This occasion, when we are met to dedicate the William Thompson Sedgwick Laboratories of Sanitary Science, is of unusual significance and satisfaction to his old students and associates. It ensures, if that were needed, the persistence of the memory of a great teacher and a remarkable leader—a pioneer in the field of Sanitary Science in it broadest aspects.

This action today by the Department of Civil and Sanitary Engineering at M.I.T. not only honors a man whose influence was compelling and far-reaching, but it confers distinction on itself and on the profession for which it stands and which it strives to advance.

As Professor Sedgwick was by choice a Teacher and regarded the teaching profession as a great calling, and for himself the greatest, it is especially fitting that one of the memorials which honor his memory and his services should be a laboratory devoted to teaching and research especially for the training of young men about to enter upon their professional work in public health or sanitary engineering; for this was one of the great objectives of his life, and the field in which he was pre-eminently distinguished and successful.

His name is connected with other memorials, — the Sedgwick Memorial Lectureship here at the Institute, in which eminent representatives of the broad aspects of biological science as well as distinguished public health men have been and will be the lecturers; the William Thompson Sedgwick Junior High School at West Hartford, Conn., near where he was born and which will perpetuate his name; and the Sedgwick Medal, awarded each year by the American Public Health Association to some outstanding worker or research man in that field of service. We, who knew him, are proud of all these, but I am sure that to have these labora-

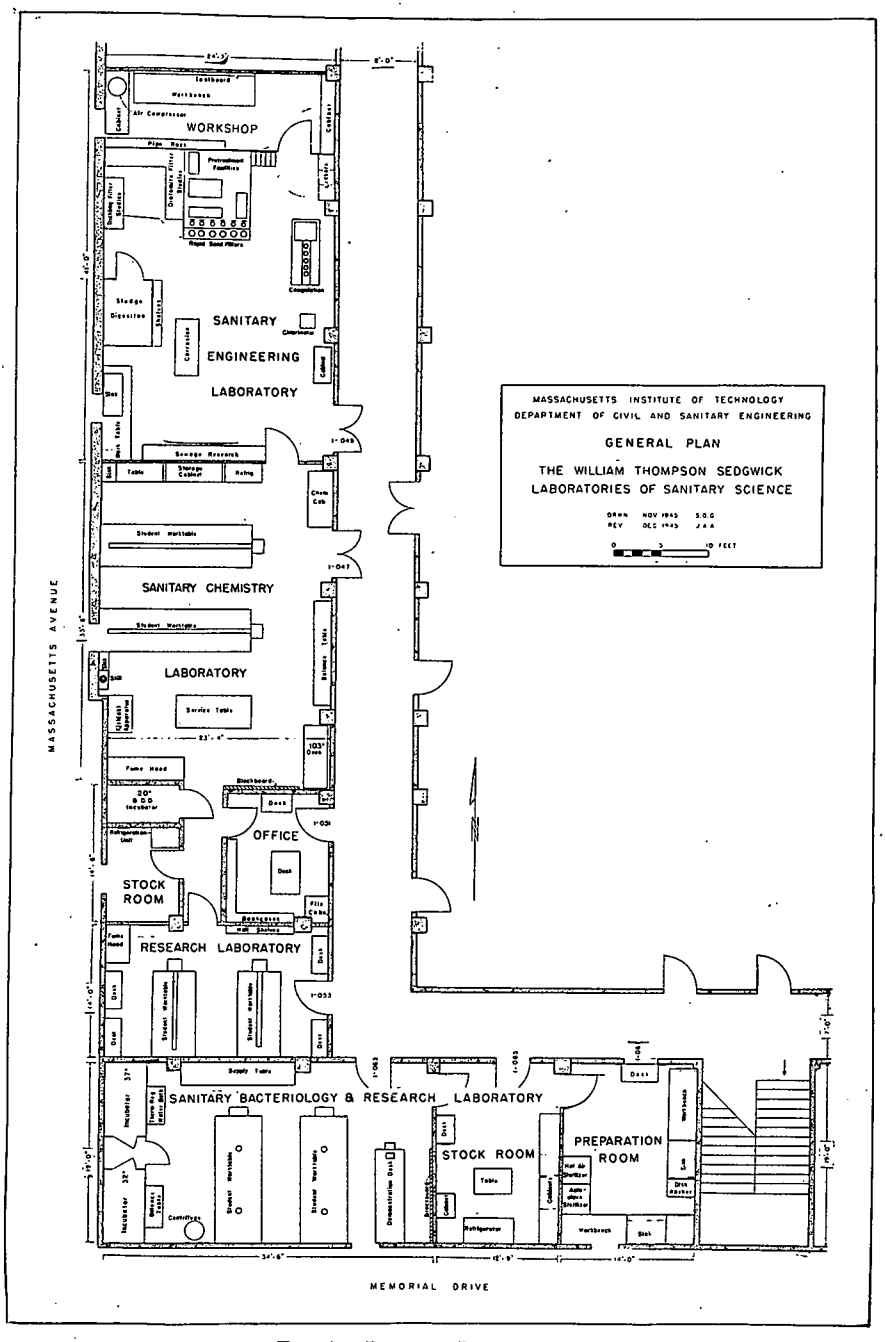


FIG. A.—PLAN OF LABORATORIES.

tories here at M.I.T. bear his name and carry on the work he loved would please him above all the others.

Great as were Professor Sedgwick's services to city, state, and nation, and to many important organizations for public service, I think it was his greatest service, and the one by which he will be most appreciatively recalled by his old students, to have been a wonderful and inspiring teacher and developer of character. As a result he brought great reputation to himself and to this institution through "Sedgwick's men". Through them and their work, his influence will be an ever-continuing one.

A rapid survey of his career is perhaps fitting at this time. He was born in Farmington, Conn., December 29, 1855, the son of a farmer. His father died when he was eight years old, and his mother was a semi-invalid thereafter. When he was ready for it, he attended the Hartford High School, living during the school months at the home of two retired and very cultured women teachers who took into their home students who came to attend this excellent school. Here he formed a life-long friendship with Mitsukuri who became the most eminent professor of zoology in Japan. At high school he hardly expected to go to college although he had hoped for possibly one year of college at Rensselaer Polytechnic Institute. The kind sisters urged and encouraged him to get a full college training, with the result that he entered the Sheffield Scientific School, and by summer work on his half-brother's farm and by tutoring Yale students in winter was able to complete his course and graduate, Ph.B., in 1877. Here began his life-long friendship with E. B. Wilson, later joint author with him of the famous General Biology. He was senior class president, and during his senior year was also assistant to Prof. Chittenden in physiological chemistry. His great interest in biology led him to think of medicine as a profession and he entered Yale Medical School, still serving as assistant, and during a second year, as substitute for Prof. Chittenden. The instruction in the medical school was disappointing as it seemed to him lacking in vitality and any appeal to imagination or research. Fortunately, at the end of two years he won a competitive fellowship at the new Johns Hopkins University and, after two happy years of study and research, took his Ph.D. in Biology in 1881. In December of that year he married Miss Mary K. Rice of New Haven, who was his admirable helpmate throughout his life. As Associate in Biology he taught for two years at Johns Hopkins. Then, in 1883, he was elected assistant professor and head of Biology at M.I.T. It was at first practically a one-man department and he was almost a trinity of professor, dishwasher, and janitor.

Excellently trained in biology, up to this time there had been nothing in his university training to stimulate special interest in public health. But at M.I.T. he met and worked with William Ripley Nichols on gas poisoning, gave the first course of instruction in New England on germs and germicides, and soon became deeply interested with Prof. Drown and Mrs. Richards in water supply, and was elected Biologist of the State Board of Health under which was brought about the establishment of the Lawrence Experiment Station. By 1888 he argued strongly for training water and sewerage engineers in the fundamentals of biology and bacteriology, and as a result, Course XI, Sanitary Engineering, was established.

Accidents sometimes lead to great advances in science. A series of outbreaks of typhoid fever in several cities and towns in Massachusetts in 1892 commanded his attention as biologist of the State Board, and these careful investigations resulted in the brilliant pioneering studies in epidemiology which have given him great

fame. Always devoted to his teaching at M.I.T., he also served as an expert in many cases, notably the Chicago Drainage Canal Case, was on the Public Health Council when the State Department of Health was reorganized, served Boston on various Commissions, begged funds to establish the Sewage Experiment Station, etc. He was one of the founders and first president of the Society of American Bacteriologists. He was a president of the American Public Health Association, and of the American Society of Naturalists, and an Exchange Professor from M.I.T. to England. He was one of the founders of the unincorporated School for Health Officers maintained by M.I.T. and Harvard, a voluntary organization which eventually led the Rockefeller Foundation to give funds for the permanent Harvard School for Public Health. He later was a member of the National Public Health Service and the International Health Board. (For complete list of his many services see: "A Pioneer in Public Health", published by the Yale University Press in 1924).

The items stated above indicate only a small part of what Sedgwick *did*. What he *was* is even more important. A man always kindly, always helpful, quick to see one's difficulties and to counsel wisely; a man of understanding as well as of knowledge. He believed in students and helped to make men of them. Tolerant and patient with all who tried to do their best, and warm and deep in his sympathy with those in trouble, poverty or sorrow, he did not hesitate to reprimand shirkers and those who tried to gain unfairly. He was a man of true culture, appreciative of poetry and art, and especially of nature. He was like a second father to many students and younger staff, and an older brother to his junior associates. He had a quick and pleasing sense of humor, and always high ideals of character and honorable dealing which he impressed on his students.

I hope it is not a breach of propriety if I mention my own personal association with Dr. Sedgwick for a quarter of a century. There is no man to whom I owe so much, not even to my own father. His influence has persisted through my whole life. In my earliest years in his department I had for him a sort of hero worship, for he had all the qualities I admired, and I felt his wisdom and great helpfulness. As the decades passed there constantly grew that mutual loyalty and strong affection, almost more than that of brotherhood, which, fortunately, men can sometimes experience. With him, I was ever conscious of what a great teacher and a great friend could and should be. Thus it gives me a deep and solemn joy to have a part in these exercises today.

In giving his name to these laboratories as they are dedicated to their high purpose, may it always be kept in mind that teaching is a sacred trust, as Sedgwick regarded it, and that the great function of the teacher is to help in the development of character, personality and integrity, or what may be called the conduct of life, as well as to impart scientific knowledge and to encourage the research spirit.

Laboratory Training and Sanitary Engineering Practice

By Arthur B. Weston

I appreciate very much this opportunity to extend to the Department of Civil and Sanitary Engineering of the Massachusetts Institute of Technology the greetings of the Division of Sanitary Engineering of the Department of Public Health which as you all well know is probably the oldest Division of Sanitary Engineering associated with any Department of Public Health in this country. Our staff was particularly pleased that its Division was considered in connection with these dedication exercises because it appreciates the importance of laboratory training in connection with any course in sanitary engineering.

Very early in the programs of environmental sanitation carried out by the Massachusetts Department of Public Health, laboratory procedure played an important part; in fact, the first scientist employed by the old State Board of Health was William Ripley Nichols, then Professor of Chemistry at the Massachusetts Institute of Technology when he investigated and then reported in the Annual Report of 1871 in connection with the presence of lead in the water supplied to the city of Boston from the then Lake Cochituate works and when he reported as to the character of the water in Mystic Pond and its tributaries. The water of Upper Mystic Lake, referred to as Mystic Pond, was used as a source of water supply for Charlestown, Somerville and East Boston. In the latter connection it is interesting to note that pollution of that pond by tannery and other wastes had reached a stage which required careful investigation and consideration. You all know that later the report of the Massachusetts Drainage Commission presented to the Governor of the Commonwealth on December 28, 1885, not only described various projects in detail giving estimates of cost relating to the pollution of streams and sewage disposal but recommended the organization of what is now our Division of Sanitary Engineering and an experimental station, commonly known as the Lawrence Experiment Station. The recommendations of that Commission resulted in what is known as Section 17 of Chapter 111 of the General Laws which provides that:

"The department shall consult with and advise the officers of towns and persons having or about to have systems of water supply, drainage or sewerage as to the most appropriate source of water supply and the best method of assuring its purity, or as to the best method of disposing of their drainage or sewage with reference to the existing and future needs of other towns or persons which may be affected thereby. It shall also consult with and advise persons engaged or intending to engage in any manufacturing or other business whose drainage or sewage may tend to pollute any inland water as to the best method of preventing such pollution, and *it may conduct experiments to determine the best methods of the purification or disposal of drainage or sewage. No person shall be required to bear the expense of such consultation, advice or experiments.* Towns and persons shall submit to said department for its advice and approval their proposed system of water supply or of the disposal of drainage or sewage, and no such system shall be established without such approval. All petitions to the general court for authority to introduce a system of water supply, drainage or sewerage shall be accompanied by a copy of the recommendation, advice and approval of said department thereon. The department may after a public hearing require a city or town or water company to make such improvements relative to any existing treatment works as in its judgment may be necessary for the protection of the public health. In this section the term 'drainage' means rainfall, surface and subsoil water only and 'sewage' means domestic and manufacturing filth and refuse."

The history of the work of our Lawrence Experiment Station is probably well known to you. Associated with this Station were many who later became recognized as some of the greatest scientists in this country. The work of this Station over a great many years has in the speaker's opinion resulted in the foundation of many of the present day approved practices in sanitary engineering science and environmental sanitation work. It is interesting to note that the old Board of Health had the benefit of the advice also of William Thompson Sedgwick, then Professor at the Massachusetts Insti-

tute of Technology and for whom the laboratory which you are dedicating today has been named. In fact, Professor Sedgwick assisted with experiments in our old State Board of Health in certain controversies in the manufacture of water gas as early as 1884. He was a consulting chemist associated with the work of our Lawrence Experiment Station as early as 1886-7 and he assisted in certain studies made by our old State Board of Health in 1887 as to the benefit of storage of water for the removal of certain bacteria known to be injurious to health. He was made a member of our first Public Health Council established in 1914 as a result of a reorganization act which became effective on July 1, 1914.

Today, perhaps even more than in the early days of the work of our Division of Sanitary Engineering, laboratory work plays a most important part. Such work in connection with water pollution control is the basis of the decision of the Sanitary Engineer. The value of this work has been emphasized during the past year of activities under a law which now gives our Department the authority to adopt rules and regulations to prevent pollution of all lakes, ponds, streams and tidal waters within the Commonwealth. The work of our laboratories relating to the treatment and disposal of wool scouring wastes and the work in connection with the operation of a pilot plant to determine new methods of shellfish treatment has been of particular value in recent months.

In carrying out your environmental sanitation programs at the Institute in connection with the laboratory which you are dedicating today, I hope that those in charge of the laboratory will feel free to make use of the data available in our Division of Sanitary Engineering and feel free at all times not only to visit and become acquainted with our Lawrence Experiment Station but free to give to the speaker, as Director of the Division of Sanitary Engineering, the benefit of advice in the conduct of our Station.

The Laboratory and Sanitary Engineering Education

By Gordon M. Fair

"Sanitation," as stated so well by Reginald Reynolds in his delightful treatise *Cleanliness and Godliness,* "has its history, its archaeology, its literature, and its science. Most religions concern themselves with it, sociology includes it within its sphere, and its study is imperative to social ethics. Some knowledge of psychology is necessary to understand its development and retardation, an aesthetic sense is required for its full appreciation, (and) economics determine, to a large degree, its growth and extent. . . ." "Whoever, indeed, would study this subject with a knowledge worthy of its magnitude must consider it from all angles and with a . . . wealth of learning. . . ."

This statement, I am sure, would have warmed the cockles of the hearts of William Thompson Sedgwick in whose memory these "Laboratories of Sanitary Science" are being dedicated this afternoon. For Sedgwick, indeed, possessed a wealth of learning and took delight in examining one by one the many facets of the science of sanitation in which he was a pioneer.

But I am not to speak about Professor Sedgwick. That has been done by Dean Prescott in words far more eloquent than I possess and out of an acquaintance that was longer and closer than that of any other of Sedgwick's pupils. May I say this, however: we, as sanitary engineers, are proud to acknowledge our debt to Sedgwick, to his sagacity——for he was very wise, to his inspiration—for he had the capacity to enthuse young men, and to his leadership—for he was unafraid of the new. Without him, our profession might, indeed, never have been created as "America's greatest gift to the nations of the world."

Rather it is my assignment in these dedicatory proceedings to state, very briefly, how laboratories such as these fit into the educational scheme of sanitary engineering. Since these rooms are to be devoted—and very fittingly so—to the science of sanitation, and not to its history, archaeology, or literature, I shall limit my remarks to a consideration of the value of laboratories such as these as instruments in the scientific training of the apprenticed and in the research of the mature.

The country over, there are but few laboratories of this kind. Fewer still are so fortunate as to include ancillary laboratories of sanitary chemistry and sanitary biology. In most educational institutions, the subject matter of sanitary chemistry and biology is taught more or less *pro forma* in *"service courses"* by members of departments of chemistry and biology who have but little contact with sanitary engineering. This is economic but, in my opinion, hardly ever satisfying. Hygeia is an exacting mistress, and devotion to her is a full-time job. Chemists and biologists who dedicate themselves to her service, furthermore, must be cast in the heroic mold, catholic in the knowledge of their science, broadly acquainted with its implications in sanitation, and (last but not least) immune to the frustrating experience of associating very closely with engineers. I have found, in general, that such men do not develop, of their own accord, in departments of chemistry and biology. They must be sought and won over. It may take a world war to do so but, once engaged in sanitary science, they seem to relish the wealth of problems that confronts them. For, as we all know, there is nothing dull about sanitary engineering.

The laboratories of sanitary engineering in this country fall, generally, into two classes: (1) laboratories that are designed to investigate and demonstrate fundamental phenomena in sanitary engineering, and (2) laboratories in which plant-scale tests are run. I need not point out that only the first can exist successfully by itself, and I am doubtful of the wisdom of ever trying to set up the second in an educational institution. There may be room, however, for a plant-scale laboratory in an engineering experiment station.

I am glad to see that the aims of the Sedgwick Laboratories are to provide opportunities for fundamental instruction and research. It has been my experience that laboratories of this kind can be a tremendous force in the development of a department of sanitary engineering. Not only will they permit students to observe, identify, and evaluate the processes—physical, chemical, and biological—that underlie sanitary engineering; not only will they offer facilities for research and the advancement of knowledge in sanitary engineering; but, most important, they will give direction to the thinking of the teachers who are associated with them and cause them to penetrate farther and farther into the theoretical aspects of their field, until sanitary science can take its rightful place in all phases of the educational program of sanitary engineering.

The Massachusetts Institute of Technology has a long and proud history in sanitary engineering. Since the tree is known by its fruits, let me now praise famous men—not alone Sedgwick, for a teacher lives not only unto himself but in the lives of many of his pupils and again in their pupils, even unto the seventy-seventh generaation. Time permits me to refer to but three of Sedgwick's students: Allen Hazen, George C. Whipple, and George W. Fuller. They graduated from M.I.T. in 1888, 1889, and 1890 respectively and became leaders in the new profession of sanitary engineering. These three were inspired by Sedgwick not merely to practice sanitary engineering as they found it, but to add immensely to its advancement. All three began as

experimentalists: Hazen at the Lawrence Experiment Station of the Massachusetts State Board of Health; Whipple at the laboratory of the Boston Water Works and in the Mt. Prospect Laboratory of the City of New York; and Fuller at Lawrence and at the experimental station at Louisville, Kentucky. All three acquired fame as consultants and designed great and useful sanitary works, many of which still serve the nation. All three wrote books that line the shelves of sanitary engineers throughout the world. All three continued to the end of their life to contribute to knowledge within their field.

As a young man, I was privileged to know them all—two of them very well indeed. I am certain, in my own mind, that much of their success was due to the faculty for critical observation, analysis, and generalization that they cultivated and acquired in the laboratory as young men. And so may I close with this wish: May the William Thompson Sedgwick Laboratories of Sanitary Science be the happy instrument for producing many of their kind.

The Laboratories and Laboratory Directors

By William E. Stanley

I. *The Laboratories*

I will take only a few more minutes of your time as we do want you to visit these laboratories which we are dedicating this afternoon. My descriptive statements will be brief.

Our "William Thompson Sedgwick Laboratories of Sanitary Science" are located in the basement of Building One, arranged in the form of an L with the largest side along Massachusetts Avenue [Figure A—Plan of Laboratories]. They include four main laboratories designated respectively, from top of the L down:

1. Sanitary Engineering Laboratory (Room 1-045)
2. Sanitary Chemistry Laboratory (Room 1-047)
3. Sanitary Chemical Research Laboratory (Room 1-053)
4. Sanitary Bacteriology and Bacteriological Research Laboratory (Rooms 1-063, 1-065 and 1-067)

As you go down the stairway in the south west corner of Building One, to the basement, and turn left you will come first to the Bacteriological Laboratory, next to the chemical research laboratory, then the sanitary chemistry laboratory, and finally to the sanitary engineering laboratory.

These four laboratories may be briefly described as follows:

1. *Sanitary Engineering Laboratory* (Figure B) contained in a room about 24 by 40 feet (984 sq. ft. floor space) equipped with the following major items:

a. *A Rapid Sand Filter,* demonstration and experimental unit, comprising 6 2-inch glass tube filters with 30 inches of sand filter media and preliminary treatment units to de-aerate the water, soil it to any desired degree in order to simulate "raw" water and then treat it with coagulants in order to produce a proper quality for filtration. (See picture in Figure B.)

b. *A Coagulation Test* unit with temperature control for studying coagulation reactions.

c. *A Chlorinator* (modern W. & T. machine) for demonstrations of the mechanism and also for feeding chlorine in experimental studies.

d. *A Sludge Digestion* unit (enclosed and ventilated with controlled temperature) for demonstrating and studying digestion of sewage and industrial waste sludges.

e. *A Small Trickling Filter* unit for laboratory scale experimental studies of high rate biological filtration of sewage and industrial wastes.

f. *Space for a Sedimentation Tank* for class demonstrations and research studies on sedimentation.

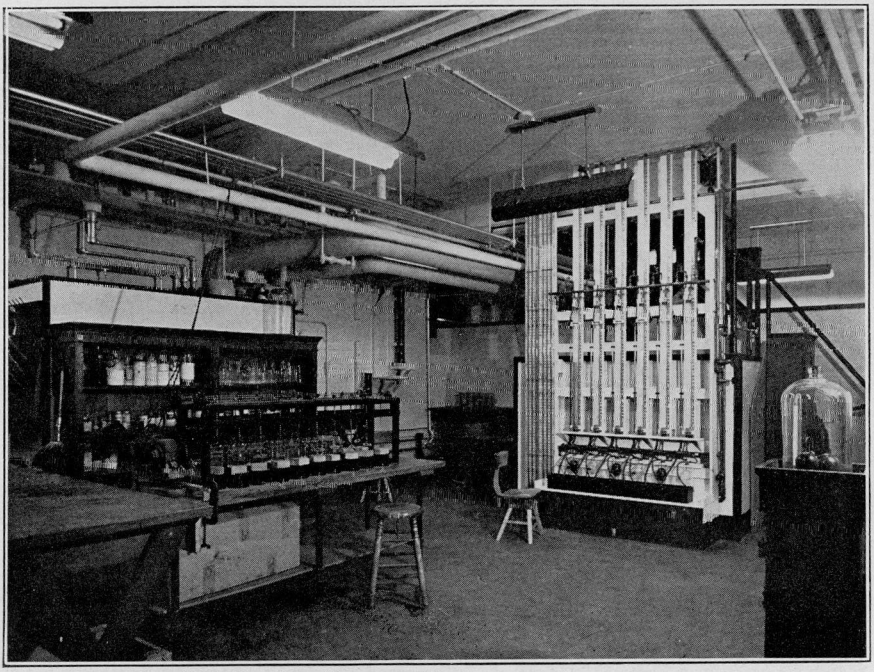


FIG. B.—THE SANITARY ENGINEERING LABORATORY.

g. *Research Projects* space—and room for experimental theses.
h. *Work Shop* at back of the laboratory, equipped with small tools, work bench and a limited number of electrical driven devices for student work.
i. *Air Compressor* and air receiver to furnish compressed air for various experimental studies.

This laboratory is also used from time to time for research studies which involve a combination of engineering and chemical aspects. For example, just now a study of Nutritional problems in Wastes Treatment, sponsored by the National Health Institute, is underway. This involves chemical analyses and also considerable air quantities and temperature control of experimental elements. It is set up in the Sanitary Engineering Laboratory directly beside the door leading into the Sanitary Chemistry Laboratory.

2. *Sanitary Chemistry Laboratory* (Figure C) contained in a room about 24 by 33½ feet (804 sq. ft. floor space) equipped with steel laboratory furniture and apparatus of latest design so as to provide students with facilities for making the standard analyses of water, sewage and industrial wastes in the modern manner. Equipment for instrumental analyses is included and will be added to as funds become available, so the advanced students and research workers are enabled to become experienced in such time saving analytical devices. Among the major items of equipment are included the following:

a. *Two Student Work Benches* with spaces for 14 students and provid-

ed with hot and cold water, air, gas, electricity and the necessary minor equipment. (Pictures in Figure C)

b. *Fume Hood* with electrically heated water bath, and electrical high temperature muffled furnace.
c. *Keldhal Nitrogen* distillation unit.
d. *Water Still.*
e. *Drying Oven* of large capacity.
f. *Balance Table* with 4 precision balances.
g. *G. E. Refrigerator* 7.5 cubic foot capacity.
h. *Cabinets* for storage of chemicals.
i. *Miscel. equipment*, such as spectrometer, potential-meter, pH units, chlorine residual units, and other items needed for standard and special tests.

The main Sanitary Chemistry Laboratory is located between the Sanitary Chemical Research Laboratory and the Sanitary Engineering Laboratory, with interconnecting doors so that all its facilities for analytical work are readily available to students and research works in all three laboratories.

Directly adjacent to and connecting into the Sanitary Chemistry Laboratory is Dr. Sawyer's office, a small storage room and a 6 by 12 feet walk-in type 20° C incubator furnished with a sensitive refrigerating unit to maintain a constant 20° C temperature.

3. *Sanitary Chemical Research Laboratory* contained in a room about 14 by 23.3 feet (326 sq. ft. floor space) equipped with two 4 space work benches, a fume hood and four desks for research workers. This laboratory connects directly into the Sanitary Chemistry Laboratory and is directly adjacent to the Sanitary Bacteriological

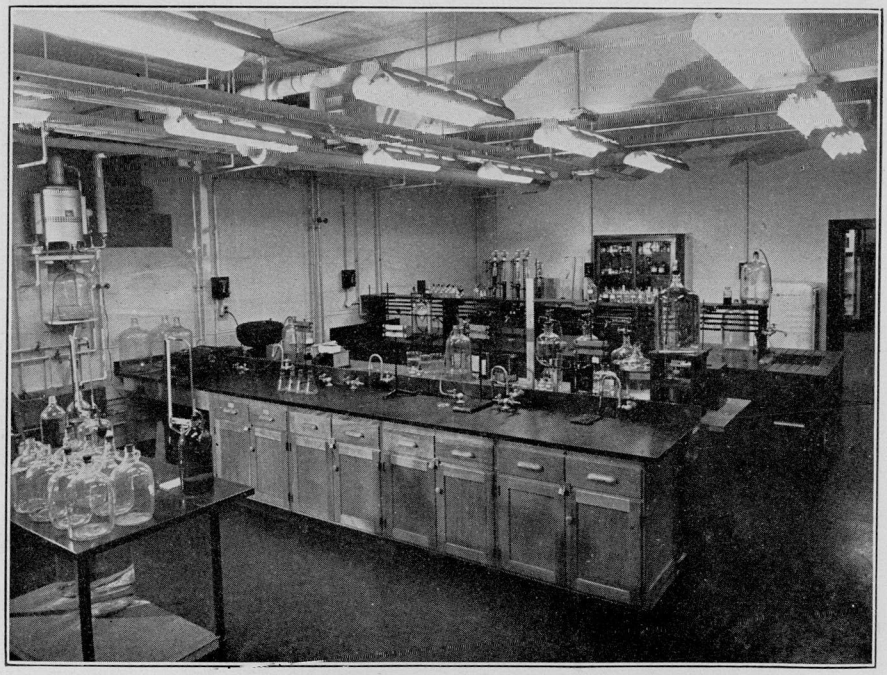


Fig. C.—The Sanitary Chemistry Laboratory.

Laboratory so research workers have ready access to the facilities of all of the William Thompson Sedgwick Laboratories of Sanitary Science.

4. *Sanitary Bacteriology and Bacteriological Research Laboratory* (Figure D) contained in a gross area of about 19 by 65 feet (1235 sq. ft. floor space) divided into (1) a preparation room about 19 by 14 feet, (2) a store room about 19 by 13 feet for sterile supplies and glass ware and (3) a main laboratory about 19 by 34½ (655 sq. ft.).

The preparation room is equipped with an electric hot air sterilizer, an auto clave provided with steam under pressure, a Charlab Junior dish washing machine, two work benches with sinks fitted with hot and cold water, electricity and gas outlets and ample storage cabinets and drawers.

The store room provides a reserve supply of sterile materials and equipment not in current use. It contains storage cabinets, a 20 cubic foot electric refrigerator for storing all prepared culture media not in use, and a desk for research workers.

The *Main Bacteriology Laboratory* (Figure D) includes commodious and attractive work benches with ample drawer space, running hot and cold water, gas and electricity outlets and facilities for 12 students at one time, an instructor's lecture and demonstration table, Quebec colony counters and tallies, facilities for quick and easy distribution of culture media, and facilities for collection of all used cultures and dishes.

At the rear of the laboratory are two walk-in type incubators, each 4 by 7 feet and provided with thermostatically

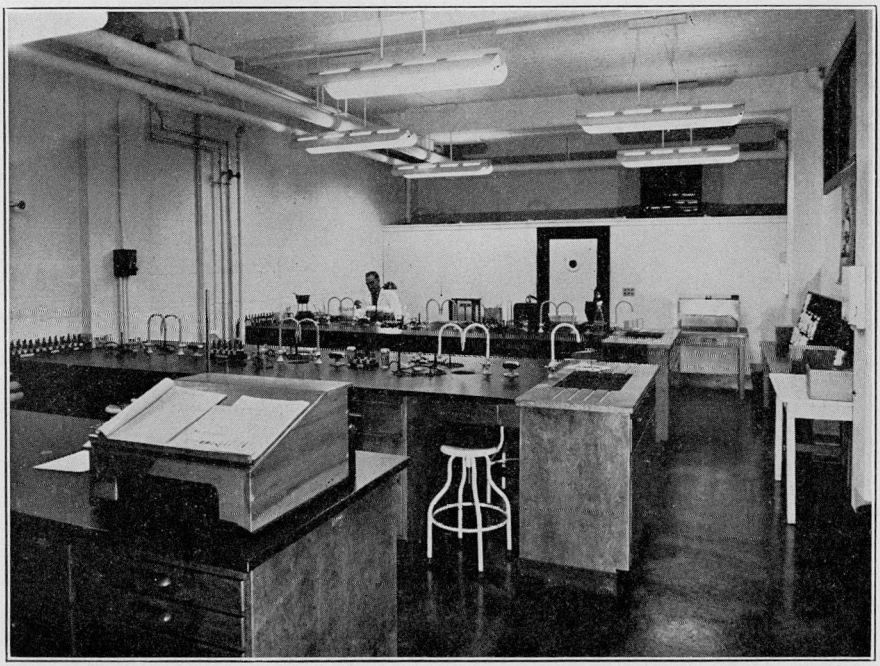


Fig. D.—The Sanitary Bacteriology and Bacteriological Laboratory.

controlled air fan type temperature regulators to maintain the incubators at 32° C and 37° C, respectively. An air lock type entrance through a small vestibule prevents any sudden change of temperature when entering either incubator. Just outside of the incubators are a portable high speed centrifuge, an analytical balance and a thermostatically controlled water bath.

The facilities available are suitable for the bacteriological analysis of water, sewage, air, shellfish, milk, foods, swab-rinse preparations from eating and drinking utensils and other requirements of a laboratory in Sanitary Bacteriology. Facilities are also available for the ordinary physical and chemical analyses made on milk supplies including the phosphatose test.

II. *The Laboratories' Directors*

The Directors of the four laboratories include:

a. Dr. Murray P. Horwood, Professor of Sanitary Science in charge of the Bacteriology and Bacteriological Research Laboratory.
b. Dr. Clair N. Sawyer, Associate Professor of Sanitary Chemistry, in charge of the Sanitary Chemistry and Chemical Research Laboratories.
c. Prof. Ariel A. Thomas, Assistant Professor of Sanitary Engineering, in charge of the Sanitary Engineering Laboratory.

Dr. Murray P. Horwood needs no introduction to most of you. He has taught Sanitary Bacteriology, Public Health Subjects and Principles of Sanitary Science for several years at M.I.T. in the Department of Biology and Public Health. In 1944 when the public health and sanitary science subjects were transferred to the Division of Sanitary Engineering, Dr. Horwood joined the staff of the Department of Civil and Sanitary Engineering, as Professor of Sanitary Science. Dr. Horwood is an M.I.T. graduate and has long been associated with educational, bacteriological and public health activities in New England.

Dr. Clair N. Sawyer, is rapidly becoming known to many of you. Like myself, Dr. Sawyer is a middle westerner by birth. He grew up and received his education largely in Wisconsin, with a short excursion into Colorado for a Master's degree. He has an enviable record as a teacher of of Chemistry at Wisconsin, Sanitary Chemistry and Biology at New York University and is now teaching Sanitary Chemistry and Industrial Wastes Treatment Processes. Dr. Sawyer is well known for his researches, including work with the activated sludge process, studies of lake pollution, and industrial waste studies. He is presently directing two researches for the National Institute of Health and a third research in the application of chlorine to industrial waste treatment.

Professor Ariel A. Thomas is a graduate of M.I.T. who went west for his Master's degree at the University of Illinois and spent five years with the Illinois State Department of Health followed by another five years as a Sanitary Corps Officer in the Army. After thus being exposed to these broadening experiences and wanderings, Professor Thomas has returned to us.

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETINGS

Boston Society of Civil Engineers

OCTOBER 15, 1947.—A joint meeting of the Boston Society of Civil Engineers and of the Northeastern Section of the American Society of Civil Engineers was held this date at Chipman Hall, Tremont Temple, Boston, Mass. Members of student chapters and civil engineering students of the New England Colleges were especially urged to attend.

Open house was held at the Society Rooms from 5:00 to 6:00 P.M., and a goodly number of students visited the rooms.

A catered dinner was held in Chipman Hall from 6:00 to 7:00 P.M. Dinner reservations from the various colleges was as follows:

Northeastern University	66
Harvard University	22
Mass. Institute of Technology	7
Tufts College	39
Rhode Island State College	26
Thayer School of Engineering	14
Worcester Polytechnic Institute	2
University of New Hampshire	3
Brown University	10

265 members and guests attended the dinner.

At 7:20 P.M., President Kinnison called the meeting to order and welcomed the students. The President thanked the student chapters and the officers of the ASCE for their assistance in the details of the meeting.

Miles N. Clair, Vice-President of the N.E. Section, ASCE, addressed the meeting and made several announcements.

The Secretary at the President's request described the emergency exits from the hall.

The speaker of the evening was Preston E. Cloud, Assistant Professor of Geology at Harvard, who gave an illustrated description of the Paricutin Volcano in Mexico. Nearly 100 colored slides and 2 rolls of movies were shown of the volcano. The volcano was of special interest because of its recent origin and its growth has been watched from its beginning.

Following the talk there were a number of questions. Rev. Daniel Linnehan of Weston College took part in the discussion.

There were 290 members and guests present at the meeting.

The meeting adjourned at 8:45 P.M.

EDWIN B. COBB, *Secretary*

NOVEMBER 19, 1947. — A regular meeting of the Boston Society of Civil Engineers was held as a joint meeting with the Surveying and Mapping Section and the Northeastern University Student Section, on this date at Northeastern University.

A catered dinner was served at 6:00 P.M. at the University Commons at which 78 members and guests and 75 students attended.

The technical portion of the meeting was held in Richards Hall, at 7:00 P.M., and was attended by 210 persons.

President Kinnison presided at the meeting and Frank L. Cheney, Chairman of the Surveying and Mapping

Section, and Clifford E. Sullivan, Chairman of the Northeastern University Section were recognized to attend to the business of their respective sections.

The President announced the recent death of members of the Society.

Frank A. Barbour who was elected a member February 17, 1892 and who died May 27, 1947.

Robert R. Evans who was elected a member February 15, 1899 and who died September 13, 1947.

Adin M. Custance who was elected a member May 20, 1936 and who died June 22, 1946.

Dana M. Pratt who was elected a member March 18, 1896 and who died August 19, 1947.

Arthur G. Robbins who was elected a member April 18, 1888 and who died October 26, 1947.

Chester W. Smith who was elected a member October 18, 1893 and who died July 22, 1947.

The Secretary read the names of recent additions to the Society.

Grade of Member. — Edmund H. Brown, Nathaniel Clapp, William R. Cuff, Stephen Haseltine,* William B. Hilton, S. Albert Kaufmann, Henry A. Kingsbury,* Benjamin A. Lekesky, Donald F. Libby,* Richard W. Logan, Ray L. Schoppe, Joseph G. Power, George R. Rich, Edward N. Smith, Stephen H. Smith, Henry E. Weiss.*

Grade of Junior. — Bernard A. Barnes,† John M. Campbell,† Donald B. Carter, Jr.,† Carl W. Eschelbach,† Eugene D. Mellish, Alfred J. Pacelli,† Kenneth E. Palmer,† Francis R. Tinsler.†

The President introduced the speakers of the evening.

Mr. Charles M. Anderson, Acting Engineer of the Massachusetts Land Court, who spoke on the "Engineering Aspects of the Land Court". His talk was illustrated by slides showing typical land court plans. Mr. Anderson outlined the engineering development of the Land Court.

Mr. Edward P. Shaw, 3rd, of Deland & Rockwood, Attorneys-at-Law, who spoke on the "Legal Aspects of Land Court as an Attorney and Examiner". Mr. Shaw outlined the legal procedure of the Land Court.

Following a discussion of the papers the meeting adjourned at 9:00 P.M.

EDWIN B. COBB, *Secretary*

DECEMBER 17, 1947.—A regular meeting of the Boston Society of Civil Engineers was held this date at Chipman Hall, Tremont Temple, Boston, Mass.

A catered dinner was served at 6:00 P.M., at Chipman Hall at which 88 members and guests attended.

President Kinnison presided at the meeting and announced the death of the following member:

Charles A. Mixer who was elected a member May 19, 1909 and who died October 24, 1947.

At the request of the President the Secretary described the fire exits from Chipman Hall.

The Secretary presented recommendations relative to the Permanent Fund and the Current Fund.

On motion duly made and seconded it was VOTED "that the Board of Government be authorized to use an amount not to exceed $2000 from the principle of the Permanent Fund for Current expenses", also on motion duly made and seconded it was VOTED "that the Board of Government be authorized to increase the Current Fund to $3000 by a transfer of $1500 from the Permanent Fund to the Current Fund".

The President stated that final action on this matter will be taken up at the January 1948 meeting of the Society.

The Secretary announced that the following had been elected to membership on this date:

Grade of Member.—Robert W. An-

*Transfer from Grade of Junior.
†Transfer from Grade of Student.

derson, Walter C. Anderson, Peter R. Bagarella, Arnold C. Blake, Dean F. Coburn, Robert E. Crawford, William E. Dobbins, Charles V. Dolan, Foster D. DuCharme, Jean M. Ducharme, Harry R. Feldman, William E. Goodwin, Phillip R. Jackson, Robert J. Kiley,* Alfred A. Lockerbie, Walter I. Lewis, Harold Mohn, Edward W. Moore, William P. Somers, Ariel A. Thomas, Gardiner E. Smith, Harold O. W. Washburn, Nathaniel N. Wentworth, Jr., Henry R. Wheeler, Jr., Forrest S. White, David Yona.

President Kinnison introduced the speakers of the evening:

Mr. William F. Ryan, Asst. Eng. Mgr. Stone & Webster Engr. Corp., who spoke on "Intersociety Cooperation for Professional Recognition", and Prof. Albert Haertlein, Gordon McKay, Prof. Graduate School of Engineering, Harvard University, who spoke on "Recent Developments in Engineering Registration", and Dean William C. White, Director of Day College, Northeastern University, who spoke on "Evening Engineering Education in the Greater Boston Area".

Following a discussion of the papers the meeting adjourned at 9:15 P.M.

There were 123 persons present at the meeting.

EDWIN B. COBB, *Secretary*

SANITARY SECTION

OCTOBER 1, 1947.—A meeting of the Sanitary Section was held in the Society Rooms, following a dinner at the Ambassador Restaurant. The meeting was called to order at 7:00 P.M. by Chairman George F. Brousseau.

Professor Stanley announced the December meeting which is to be held at M.I.T. in conjunction with the dedication of the "William Thompson Sedgwick Laboratories of Sanitary Science".

Mr. A. E. Griffin of Wallace and Tiernan, Inc., was the speaker of the evening, taking the subject of "Chlorine in Waters and Wastes".

Attendance was 49 at the meeting and approximately 35 at the dinner.

KENNETH F. KNOWLTON, *Clerk*

STRUCTURAL SECTION

OCTOBER 8, 1947.—Following dinner at the Smorgasbord, a joint meeting of the Structural Section and the Transportation Section was held at the Society Rooms, 715 Tremont Temple.

The meeting was called to order at 7:15 P.M.

Chairman Wyner asked Chairman Hankinson of the Transportation Section if he had any business to bring before his section. Mr. Hankinson having no business to report, the clerk's report of the last meeting of the Structural Section was read and approved.

The report of the Executive Committee action on the motion to consider a more appropriate name for the section was read. Following a brief discussion it was moved and seconded that "the Board of Government be petitioned to change the name of the Designers' Section to Structural Section". The motion carried.

The Chairman then introduced the speaker, Mr. A. L. Delaney of the Portland Cement Association who spoke on the uses of pressure grouting in railroad bed stabilization. The talk was illustrated by a moving picture of grouting operations on four of the nation's railroads. Mr. Delaney then discussed the use of pressure grouting in other phases of engineering such as stabilization of dam embankments, reclaiming old stone masonry, building foundations, graving docks, etc.

The meeting adjourned at 8:55 P.M.

Fifty-four members and guests attended.

ROBERT W. MOIR, *Clerk*

NOVEMBER 12, 1947.—Following dinner at the Smorgasbord, a meeting of the Structural Section was held at the Society Rooms, 715 Tremont Temple.

*Transfer from Grade of Junior.

Chairman Wyner called the meeting to order at 7:21 P.M.

The minutes of the last meeting of the Section were read and approved.

Chairman Wyner then introduced the speaker Mr. Wm. H. Owens, Chief Engineer of Cram & Ferguson Co., Boston, who spoke on "Structural Aspects of Mechanical Equipment in Modern Office Buildings". The speaker described the structural problems encountered in providing for elevators, escalators and other mechanical equipment, including air-conditioning and distribution systems in the new John Hancock Building and other modern office buildings in the city of Boston. The talk was illustrated by lantern slides.

After a short discussion period, the meeting was adjourned at 9:05 P.M.

Thirty-nine members and guests were present.

ROBERT W. MOIR, *Clerk*

HYDRAULICS SECTION

MAY 7, 1947.—A meeting of the Hydraulic Section was held in the Society Rooms, 715 Tremont Temple, following a dinner at the Ambassador Restaurant.

In the absence of Chairman Hooper, the meeting was conducted by Vice-chairman John G. W. Thomas. The minutes of the last meeting were read and accepted.

The speaker of the evening was Howard M. Turner who gave a very interesting illustrated paper on "Repairs of a Dam at South Barre, Mass." A short discussion followed and the meeting adjourned at 9:00 P.M.

Forty-four members and guests attended the meeting.

JAMES F. BRITTAIN, *Clerk*

APPLICATIONS FOR MEMBERSHIP

[January 20, 1948]

The By-Laws provided that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as srtictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of fifteen (15) days from the date given.

For Admission

ETHEL H. BAILEY, Brookline, Mass. (b. August 18, 1896, Houlton, Maine). 1927-1932, Rutgers University Extension, Newark, New Jersey; December 1928 Law Clerkship Reg. with Clerk N. J. Supreme Court; 1929-1930, Newark Technical School, Newark, N. J.; 1929-1930, New School for Social Research, New York City; 1938-1939, Montclair State Teachers College, Montclair, N. J.; 1940, Columbia University Summer School, New York City; 1939, Newark University Newark, N. J. Experience, 1917-1918, inspector Airplanes and Engines, A.P.S.; 1919-1921, assistant to Head of Spec. & Material Section Aircraft Division, Washington, D. C.; 1922-1924, Aero Mat. Engr., L. E. Eng. Corp., College Point, N. Y.; 1924-1926, Engr. Research Dept. Soc. Automotive Engineers, New York City; 1926-1928, M.E. Art Metal Works, Newark, N. J.; 1929-1934, M.E. General Electric Company, Bloomfield, N. J.; 1934-1936, M.E.

New Jersey State E.R.A., Newark, N. J.; 1936-1942, M.E. Montclair Public Library, Montclair, N. J.; 1942-1943, Adm. Officer, Signal Corps Radar Lab., Belmar, N. J.; 1943-1945; M.E. Raytheon Mfg. Company, Waltham, Mass.; 1945-1946, M. E. Buck Printing Company, Boston, Mass. Professional Organizations, Fellow A.A.A.S. 1925; Member A.S.M.E. 1926, 1935; Member N.S.P.E. 1935; Member Mass. Soc. P.E. 1947. Registered—New Jersey, M.E. 1935; P.E. 1938. Refers to *E. B. Cobb, E. F. Childs, K. R. Kennison, R. W. Moir.*

THOMAS A. BERRIGAN, Westwood, Mass. (b. May 29, 1895, South Boston, Mass.) Graduated from Mass. Institute of Technology in 1916, B.S. degree in Sanitary Engineering. Experience, 1916, Assistant Bridge Inspector, Fitchburg Division, B. & M. R.R.; 1917-1918, Structural Steel Draftsman and later Resident Engineer on construction, Bureau of Yards and Docks, Washington, D. C.; 1919, Estimator and Reinforced Concrete Designer, Truscon Steel Company; 1920, Designing Engineer (structural features), Lockwood Green Co.; 1921-1925, conducted contracting business; 1925-1934; Designing Engineer and later Resident Engineer on Subway, Tunnel & Building Construction, Boston Transit Department; 1935-1942, Assistant Engineer, Senior Engineer, and Sanitary Engineer, Metropolitan District Commission; 1942-1945, Lt. Comdr. in Civil Engineer Corps, U. S. Navy, with principal duty as Executive Officer for Public Works, First Naval District; 1945-1948, Director and Chief Engineer, Sewerage Division, Metropolitan District Commission; and Chairman of the Merrimack River Valley Sewerage Board. Refers to *T. R. Camp, M. N. Clair, W. E. Merrill, A. D. Weston.*

CHARLES C. CAIN, Medfield, Mass. (b. July 11, 1906,, Sherborn, Mass.) Attended Medfield High School and one year at Northeastern University plus several evening courses. Experience, 1925 to date, with the Norfolk County Engineering Department. Present position First Assistant to the County Engineer. Registered Professional Engineer and Land Surveyor. Refers to *H. A. Kingsbury, F. H. Kingsbury, F. L. Cheney, K. McIntyre, E. Worthington.*

JOHN J. CASHMAN, JR., E. Milton, Mass. (b. November 12, 1908, Boston, Mass.) Attended Grammar and High School in Milton, Mass., and graduated from Chauncey Hall School in 1929. Attended Mass. Institute of Technology from 1929 to 1931 and majored in "Building Construction". Also completed a two year Building Construction Course at Lowell Institute, Boston, 1931-1933. Experience, 1926-1927, with the Massachusetts Metropolitan District Park Commission as rodman and transitman; 1928-1930, with T. Stuart & Sons Company of Watertown as assistant field engineer on the North Station Development Project in Boston; 1932-1935, employed by the M & R Construction Company of Boston, estimated cost of construction and supervised construction of several timber wharves and trestles. 1936-1942, employed by the Works Progress Administration, first as field engineer and later as district director of operations and responsible for engineering supervision of several large projects in So. Eastern Massachusetts. Enlisted in the U. S. Navy as a CPO in 1942 and received commission in October, 1942. Served with the Construction Battalions in the Pacific Area and returned to inactive duty in 1946 with rank Lt. Cdr. in the Civil Engineer Corps. 1946 to date with Fay, Spofford & Thorndike, Engineers, Boston, assigned as Resident Engineer on a Timber Wharf job in Portland, Maine, then assistant project engineer on an Army development project in Alaska. At present Resident Engineer on job in

Waterville, Maine, involving construction of a concrete dam on the Kennebec River. Refers to *C. A. Farwell, F. L. Lincoln, R. A. Lofgren, M. H. Mellish.*

WARREN E. DAVIDSON, Baton Rouge, La. (b. February 8, 1914, Temple, New Hampshire). Graduated from Northeastern University in 1938, Civil Engineering. Experience, 1937-1940, junior engineering aid for Met. Water Supply Commission, Boston. Left at completion of job to accept position at Bethlehem Steel Company, Fore River Shipyard for Expansion Program before War. Worked on Ways, Piers, Building Construction. After completion of construction accepted position as coordinator in Hull Department in charge of coordinating prints; bills of material for ship construction. 1943, Ensign in U. S. Navy, served from December, 1943 to June, 1946, Assistant Industrial Managers Officer in ship repair; September, 1946 to May, 1947, field engineer for Leo A. Fuller Construction Company. At present employed by Stone & Webster Engineering Corp. as field engineer on construction of Oil Refinery in Baton Rouge, La. Refers to *C. O. Baird, E. A. Gramstorff, C. S. Ell, H. Campbell.*

FRED M. DODGE, Newtonville, Mass. (b. February 12, 1906, Bridgton, Maine). Graduate of the University of Maine with B.S. degree in Civil Engineering, in 1928. Experience, 1928-1931, American Telephone & Telegraph Company, headed field party making survey for new cable route. Prepared plans and material requirements for relocation of cable lines due to highway construction. Made field trips to obtain data for the preparation of records of existing cable routes in New York State; 1942-1946, U. S. War Department, Boston, Mass., prepared engineering plans, specifications, and material requirements for telephone cable systems and modifications and extensions of same. Made surveys to locate underground cable routes so that necessary easements and rights of way could be obtained. For a period of two years had charge of the record section of the Signal Corps which consisted of a personnel of eight people. Maps and plans were prepared of all cable routes and switchboard rooms in the First Service Command. Was necessary to lay out all the work, give instructions when necessary and make a final check before submitting the drawings to the Chief Signal Officer in Washington, D. C.; April, 1946 to February, 1947, Rowland H. Barnes Company, Newtonville, Mass., prepared plans and sketches of surveys made in the field. Ran transit on surveys; February, 1947 to June, 1947, Singmaster and Breyer, New York City, prepared drawings of freshwater, sewer, and acid pipe lines. Prepared bills of material from layout drawings. Made survey and prepared plans of four large adjoining buildings. The plans included all pipes within the buildings and their elevation at all points. Made several surveys for proposed routes for pipe lines. June, 1947 to date, Fay, Spofford & Thorndike, Boston, prepare plans showing the construction of manholes, utilidors, including the layout of all piping within. General piping drafting. Refers to *H. L. Crocker, C. A. Farwell, F. L. Heaney, M. H. Mellish, W. L. Hyland.*

WILLIAM G. DYER, Braintree, Mass. (b. October 28, 1911, Braintree, Mass.) Graduate of Northeastern University in 1936. Experience, 1936 to 1942, employed by Ernest W. Branch, Inc., as Civil Engineer Surveyor; 1942-1943 (13 mo.) with the Pitometer Company, New York City, as field engineer on water waste survey of water distributon system, City of Philadelphia, Pa.; 1943 to 1946 (3 1/3 yrs.) with War Department First Service Command Headquarters, Boston, as sanitary engineer; head of water and sanitation section; chief of utilities branch, final

assignment as Chief of Utilities included supervision of sixteen professional engineers; 1946 to present, with Fay, Spofford & Thorndike, Boston, as sanitary engineer on design of sanitary facilities at War Department Installations in the Alaska area. Refers to *F. L. Flood, F. L. Heaney, M. H. Mellish, G. N. Watson.*

MANUEL A. FERNANDEZ-ESTE, Boston, Mass. (b. June 7, 1921, San Fernando de Apure-Apure State, Venezuela). Graduated from Universidad Central de Venezuela, Caracas, Venezuela, July, 1946, as "Doctor en Ingenieria" (Civil Engineering degree). Experience, January, 1946 to April, 1947, self employed on planning, design and supervision of construction of buildings in Caracas and Villa de Cora, Aragua State; also made plans for residential developments in Caracas. In the Spring of 1947 in conjunction with the Institute of Inter-American Affairs came to the United States to study Hospital Planning. July, 1947 to November, 1947, was stationed in the Hospital Facilities Branch, U. S. Public Health Service, Washington, D. C., observing hospital planning technique. December, 1947 to date with Curtin and Riley, Boston, Mass., assisting in the preparation of preparing hospital drawings. Refers to *C. M. Kelley.*

PUCCIO S. GAETANO, W. Roxbury, Mass. (b. December 24, 1885, Tiriolo, Italy). Four years High School in Tiriolo, Italy; 2 years college Reale Leceo Cantanzaro, Italy, graduating in 1903. Experience, 1903-1907, worked as journeyman stone mason; 1907 to February, 1913, self contracting in Flushing, L. I., New York; March, 1913 to August, 1918, planning, estimates and reports, Architectural Concrete Products Company; August, 1918 to December, 1919, field engineer, Nestles Food Company, New York City; January, 1920 to December, 1933, self engineering and contracting, Oneonta, New York; December, 1933 to December, 1940, construction supt., Federal Works Adm.; August, 1941 to February, 1942, Times Square Mission Engineer, New York City; March, 1942 to November, 1942, assistant field engineer, U. S. Army, Syracuse, New York district; December, 1942 to March, 1944, magneto inspector, Scintilla Corp., Sidney, New York; April, 1944 to November, 1944, construction engineer, Linn Corp., Oneonta, New York; November, 1944 to August, 1945, assistant mechanical engineer, Holtzer-Cabot, Boston, Mass.; August, 1945 to February, 1947, self employed contracting, Boston, Mass.; March, 1947 to May, 1947, engineer, Devenco Corp., New York City; at present with Fay, Spofford & Thorndike, Boston, Mass., assistant engineer, reports and estimates. Refers to *W. L. Hyland, M. E. Kelley, M. H. Mellish, E. N. Tashian.*

PAUL C. GRUETER, Milton, Mass. (b. October 8, 1910, Boston, Mass.) Graduated from Mass. Institute of Technology in June, 1934, with degree of Bachelor of Science in Civil Engineering. Experience, June, 1934 to March, 1935, employed on several surveying, engineering and construction jobs; April, 1935, to June, 1936, Technical Assistant in the Hydraulics Laboratory of M.I.T., constructing, operating and preparing report on Cape Cod Canal Model; June, 1936 to Sept., 1940, U. S. Engineer Office, Boston, Mass., junior and assistant engineer, assistant to Chief of Special Studies Section (model studies tidal and current studies and corrosion of metals); Sept., 1940, to June, 1944, U. S. Engineer Office, Boston, Associate Engineer. Chief of Special Studies Section in charge of studies of tidal and current phenomena in rivers and harbors, investigation of corrosion of metals in sea water. Also Chief of Specification and Report Section, responsible for preparation of specifications for fortifications, airfield, cantonments and de-

fense construction. Prepared and edited reports on pavement studies of airfields; June, 1944 to July, 1946, U. S. Army, Post Engineer at Harbor Defenses of New Bedford. Responsible for maintenance, construction and operation of buildings utilities and other facilities; August, 1947 to date, U. S. Engineer Office, Boston, engineer, P.A. prepare specifications, estimates and reports for airfield construction and reports for special works, i.e. materials corrosion investigation and frost investigations. Refers to *E. F. Childs, H. S. Perdikis, C. A. Moore, H. I. Wyner, W. W. Zapolski.*

DONALD N. HALLOCK, Quiriquire, Venezuela. (b. August 31, 1922, McAlester, Oklahoma). Graduated from Agriculture and Mechanical College, May, 1943, B.S. degree in Civil Engineering. Experience, May, 1943 to June, 1946, Lt. (jg) CEC, USNR, attached to USNCB 140 for 8 months and to USN Underwater Demolition Team 8 for 21 months. June, 1946 to June, 1947, M.S. Sanitary Engineering, Harvard University; June, 1946 to date, District Sanitary Engineer, Creole Petroleum Corp., Quiriquire Camp, Caripito, Edo. Monagas, Venezuela. Refers to *G. M. Fair, H. A. Thomas.*

ARTHUR T. IPPEN, Belmont, Mass. (b. July 28, 1907, London, England). Diplom-Ingenieur with high honors in Civil Engineering at the Technical University of Aachen in 1931; M.S. in Civil Engineering at California Institute of Technology, Pasadena, California in 1935; Ph.D., California Institute of Technology in 1936. Experience, 1929, investigator and designer for large ground-water pumping plant for the Municipal Water Works, Aachen; 1932, teaching and research assistant, Geodetics and Surveying, Technical University of Aachen; 1932-1933, Exchange-fellow of the Institute of International Education. Hydraulics and Civil Engineering at the State University of Iowa; 1934-1936, Teaching and Research Fellow, Hydraulics Department, California Institute of Technology; 1936, Research Fellow in charge of valve testing for Metropolitan Water District of Southern California at Hydraulic Machinery Laboratory, California Institute of Technology; 1936-1938, Research Fellow in charge of high velocity flow research on open channels for Los Angeles County Flood Control District at Hydraulic Structures Laboratory, California Institute of Technology. Instructor in Hydraulics, California Institute of Technology; 1938-1939, Instructor in Hydraulics, Department of Civil Engineering, Lehigh University, Bethlehem, Pa., in charge of Hydraulic Laboratory and Hydraulic Model Testing; 1939-1945, Assistant Professor of Hydraulics, in charge of Hydraulic Laboratory and Hydraulic Research, Lehigh; 1943-1946, in charge of extensive testing program for Ingersoll-Rand Company, Phillipsburg, N. J., on "Influence of Viscosity on Centrifugal Pump Performance", Lehigh; 1946 to date, Associate Professor of Hydraulics, Dept. of Civil and Sanitary Engineering, Massachusetts Institute of Technology; in charge of Division of Hydraulics, including Instruction and Hydraulic Laboratory. Refers to *J. B. Babcock, T. R. Camp, D. W. Taylor, J. B. Wilbur.*

PHILIP E. JOHNSON, Reading, Mass. (b. March 1, 1915, Milan, N. H.) Attended Berlin High School 1929 to 1933; University of New Hampshire from 1933 to 1937, receiving B.S. degree in Civil Engineering in June, 1937. Experience, June to September, 1937, with the City of Berlin, N. H., as chief of party for surveying and preparing a revised city map. September, 1937 to February, 1938, U. S. Engineer Sub-Office, Concord, N. H., in the Soils Laboratory; November, 1938, accepted present position with the U. S. Engineer Office, Boston, Mass. Duties with

the engineers have been as follows: hydraulic design of flood control dams and related appurtenances, design of storm and sanitary drainage system for airports and army cantonments, studies on economics analysis for justification of flood control projects, hydraulic and hydrological studies of stream flow, run-off, precipitation, frequency, etc., and several special studies such as evaluation of power benefits and malaria control. In addition to the preceding office duties was office engineer and later resident engineer for three years, 1941 to 1944, on two large military construction projects at Boston, Mass., and Portland, Maine, harbors. Present position is Engineer (Hydraulic) with the Department of the Army, Corps of Engineers, Boston, Mass. Refers to *E. F. Childs, K. Eff, M. H. Mellish, J. J. Scanlon, H. I. Wyner.*

HAROLD V. KALER, Foxboro, Mass. (b. February 8, 1896, Boston, Mass.) B.S. degree, Mass. Institute of Technology in 1918. Experience, U. S. Navy, World War I, in charge repair work on ex-German vessels. Miscellaneous contracting design and survey work. Designed and rebuilt plant for Boston Dry Dock Company at Chelsea. In charge repair work. Made design and overall plan for the Navy Yard Annex at South Boston and detail design of sheet pile bulkhead and piers. Wrote a number of "Manufacturing Manuals" for U. S. Army Fire Control Division to guide manufacture of precision instruments. Design and in charge of construction of various warehouse, foundry and factory buildings. Made estimates for R.F.C. on several projects. Design work and in charge of construction of various marine railway dry docks, piers, etc. At present with Crandall Dry Dock Engineers on design and construction. Refers to *J. B. Babcock, 3rd, J. S. Crandall, C. A. Farwell, R. H. Lindgren.*

CHARLES V. KEANE, Brighton, Mass. (b. July 19, 1906, Newton, Mass.) Graduated from Mechanics Arts High School in 1925; attended University Extension course on surveying at Mass. Institute of Technology, evenings 1934-35. Experience, July, 1929 to January, 1931, Commonwealth of Massachusetts Public Works Dept., Highway Division, junior engineering aide on preliminary survey; September, 1934 to December, 1935, Boston Transit Commission, transitman running lines and grades; December, 1935 to March, 1940, City of Boston Planning Board, Chief of Party. Made triangulation surveys and did necessary computations in conjunction with survey; March, 1940 to December, 1940, New England Foundation Company, Boston, field engineer. Made preliminary survey. Did all layout, took soundings and kept progress records for the construction of wood piers at the South Boston Naval Annex; December, 1940 to October, 1941, Matthew Cummings Company, Boston, field engineer in charge of layout, cost and progress for the construction of 9 story warehouse for U. S. Navy at South Boston; October, 1941 to November, 1942, Ray B. Rendle, East Boston field engineer responsible for all layout, soundings, progress and materials in building of wood piers quay wall and sewer lines; May, 1942 to December, 1942, General Ship and Engine Works, East Boston, field engineer in charge of construction of ship's ways and buildings. Established lines for construction of ships; December, 1942 to August, 1945, Bethlehem Steel Company, East Boston, draftsman, ship drafting; August, 1945 to January, 1946, Thomas O'Connor Company, Cambridge, field engineer on construction of paper factory. Responsible for all layout, made quantity surveys, cost and progress records; January, 1946 to July, 1946, Walter Campbell, Boston, draftsman, making measured drawings, planning alterations; July, 1946 to December, 1947, C. J. Maney, Somer-

ville, engineer draftsman, doing layout of building and utilities making "as built" drawings; April, 1947 to present, Fay, Spofford & Thorndike, as senior draftsman in the Sanitary Department. Refers to *G. C. Douglas, W. L. Hyland, F. L. Heaney, M. H. Mellish.*

WILLIAM H. KRAPOHL, Boston, Mass. (b. September 16, 1899, Meriden, Conn.) Northeastern University, 1915 to 1919; Wentworth Institute, 1944; Experience, August, 1937 to August, 1940, Albert B. Franklin, Inc., Boston, designing engineer; September, 1940 to December, 1941, Zone Constructing Quartermaster, Army Base, Boston, Mass. senior engineer; December, 1941 to January, 1943, N. J. Engineering Department, Boston, Mass., senior engineer; January, 1943 to May, 1945, E. B. Badger & Sons Company, Boston, Mass., in charge of all heating, ventilating and air conditioning; 1945 to present, A. & K. Engineering Company, Inc., own business; November, 1946 to date, Fay, Spofford & Thorndike, mechanical engineer; Member of American Society of Heating and Ventilating Engineers; Registered Mechanical Engineer, Massachusetts 1796. Refers to *H. L. Crocker, F. L. Heaney, W. L. Hyland, M. H. Mellish.*

KAZIMIERZ J. KRAWCZYK, Boston, Mass. (b. June 26, 1910, Lodz, Poland). Technical University, Lwow, Poland (Civil Engineering) Master of Science, 1934. Experience, during study, 1930, Krakow waterworks construction of concrete foundations for oil tanks, narrow gauge railroad track roads (3 mo.); 1931, Highways and Bridges Dept. of District Luck, construction of bridges and roads (3 mo.); 1932, War Department, District Lublin, construction of barracks and reinforced concrete shelters (3 mo.); 1933, construction of dwelling house in Lwow (4 mo.). Since 1934, design and supervision of construction work, structural steel, reinforced concrete, timber structures, industrial buildings, foundations, appraisals, consulting work; 1937 to 1939, commissioner for building (structural) for District Warsaw, Poland; 1939 to 1945, Polish Army; 1945 to 1946, Civil Enginer with U. S. Army Engineers in Germany. Reconstruction work. Design and supervision of buildings, bridges, investigations of structures and reports; 1946 to 1947, E. B. Badger Company, Boston, structural designer; at present, assistant engineer with Fay, Spofford & Thorndike, Boston (structural department). Registered Professional Engineer with Commonwealth of Massachusetts. Student at University of Extension, Massachusetts. Refers to *G. C. Douglas, F. L. Heaney, W. L. Hyland, F. L. Lincoln, M. H. Mellish.*

ROBERT S. LOOMIS, Brookline, Mass. (b. August 8, 1926, East Hartford, Conn.) Graduated from the M.I.T. in Building Engineering and Construction in June, 1946. Experience, June, 1946 to September, 1947, worked with R. W. Loomis, Engineer in Windsor, Conn., on structural design. September, 1947, reentered Mass. Inst. of Technology to study for Master's degree and upon graduation intend to return with R. W. Loomis, Engineer. Refers to *W. M. Fife, D. Peabody, Jr., D. W. Taylor, W. C. Voss.*

FRANK MARCUCELLA, Medford, Mass. (b. September 15, 1900, Caserta, Italy). Graduated from Mass. Institute of Technology in 1927, C.E. degree in Civil Engineering. Experience, 1927-1929, Metropolitan District Water Commission; 1929-1932, George A. Fuller Company as Construction Engineer on the United Shoe Machinery Bldg., and the Worcester Auditorium; Construction Superintendent for Mass. Institute of Technology dormitory, kitchen, dining bldg., Tewksbury State Hospital, Southwick Memorial and Waterman Memorial at University of Vermont, Cadet Armory at West Point,

New York and supervised design and construction of a 3 year program for Gen. Jacob L. Evers, then athletic director at West Point. Construction Superintendent at Watertown Arsenal Warehouse Bldg., Squantum Naval Air Station, Dravo Shipyard at Wilmington, Delaware, Housing facilities at Bedford Airport, Bath housing project at Bath, Maine, U. S. Navy for 32 months in CEC Marycliff Academy, Winchester Nurses Home and Medical Bldg., at Vermont State Hospital, which brings me up to date. Have Building License for City of Boston work and was registered as a professional engineer last year. At present am General Manager and Vice-President of Volpe Construction Company, Inc. Refers to *J. B. Babcock, E. B. Cobb, W. S. Mariner, J. B. Wilbur.*

FREDERICK C. MERRIKIN, Brockton, Mass. (b. July 27, 1910, Brockton, Mass.) Graduated from Northeastern in 1936 with B.S. degree in Civil Engineering. Experience, with U. S. Engineer Office, Boston, 1936 to 1937; J. G. White Corp., New York, September and October, 1937, on Flood Control Hydraulics and Hydrology. With Employers Group Ins. Co., Boston, as claims adjuster November, 1937, to August, 1938; with Constructing Q.M., Fort Devens, Mass., August, 1938 to November, 1938, as engineering aide on topographic surveys and foundation construction. Employed by U. S. Engineer Office, Boston, November, 1938 to January, 1944; U. S. Army Sanitary Corps Officer, January, 1944 to April, 1946; returned to U. S. Engineer Office, Boston, May, 1946 to date. All work with U. S. Engineer Office in flood control hydrology and hydraulics, airport drainage and utilities for army camps and airfields. Refers to *E. F. Childs, K. Eff, M. H. Mellish, H. I. Wyner.*

CHARLES L. MURPHY, Milton, Mass. (b. February 15, 1909, Boston, Mass.) Completed four year course structural design, Franklin Technical School, Boston, in 1931; Surveying, Mass. University Extension (1935); Machine Design, Franklin Technical School (1938); Structural and Reinforced Concrete Design, Mass. University Extension (1940); Airplane Design, War Defense, Northeastern University (1941); Calculus (W.D.) Mass. Institute of Technology (1942); Hydraulics, Lincoln Tech. (1944); Eng. Structures, Lincoln Technical (1945). Experience, 1929-1933, 1937-1939, James A. Cotter Company, Sanitary Engineers, Boston, Junior Engineer on Design; 1939, Charles L. Thompson, Sanitary Engineer, Boston as Junior Engineer; 1939-1947, U. S. Engineers, Providence; 1947 to date Fay, Spofford & Thorndike, Boston, as Sanitary Engineers. Refers to *H. L. Crocker, L. Hersum, F. L. Heaney, W. L. Hyland, M. H. Mellish.*

STEWART H. NEWLAND, Newtonville, Mass. (b. October 5, 1905, Wilmette, Illinois). Graduated from the Mass. Institute of Technology in 1928, received B.S. degree in Civil Engineering. Experience, following graduation worked for United Engineering, Inc., the Engineering Dept. of the Commonwealth & Southern Power Company in Saginaw, Michigan and Blue Ridge, Georgia, on the construction of steam and hydro electric plants until the fall of 1932. With Fitzhenry-Guptill Company, pump manufacturers, E. Cambridge, Mass., until the spring of 1933; with Freeport Sulphur Company, New Orleans, La., on sulphur plant construction in S. Louisiana until the winter of 1933-34; with the Government in Louisiana until the winter of 1934-35; with the Lousiana State Board of Health, as field engineer until January, 1941; since that time with Wallace & Tiernan Company in West Texas and New Mexico, as district sales engineer. At present employed by Wallace & Tiernan Company, Inc., as Manager of the Boston Office. Refers to *A. J. Bur-*

doin, T. R. Camp, F. S. Gibbs, J. B. Wilbur.

GEORGE L. NEWMAN, Boston, Mass. (b. October 26, 1895, Boston, Mass.) Sept., 1915 to June, 1917, Wentworth Institute, Building Superintendance Certificate; Sept., 1915 to June, 1917, Lowell Institute, Building Foreman Certificate; Sept., 1921 to June, 1922, Lowell Institute, Mechanical Industrial Foreman Certificate; Sept., 1923 to Feb., 1924, Harvard College, Architectural Design of Buildings; Sept., 1923 to June, 1924, Boston University Evening School, real estate law and practice; Sept., 1924 to June, 1925, Northeastern University Evening School, structural engineer and foundations, certificate for 3 year course. Experience, 6 years draftsman, architect and engineer; 6 years specification writer; 2½ years sales engineer; 2 years structural engineer with the following firms: J. McFarland, Architect, Boston; Monks & Johnson (firm extinct) Engineers, Boston; Stone & Webster Engineering Corp., Boston; A. Rosenstein, Architect, Boston; Parker Thomas & Rice, Architects, Boston; J. W. Beale & Son, Boston; Coolidge, Shepley Bullfinch & Abbott, Architects, Boston. Ten years State Building Inspector. Refers to *B. S. Brown, A. G. H. Dietz, H. G. Protze, O. Horovitz, T. R. Worcester, H. Weiss.*

ROWEN C. PARKER, Winchester, Mass. (b. July 10, 1887, Nashua, New Hampshire). Graduated from Woburn High School in 1906. Entered Harvard College taking engineering subjects and also completed the summer courses at the Harvard Engineering Camp. Graduated from Harvard College with A.B. degree in 1910. Experience, Stone & Webster, Boston, Mass., left after three years to take an opening in a bank in Shreveport, La. After overseas service in World War I was engaged for few months on a bridge construction job (material inventories and costs) in Shreveport, La. Returning North, studied accounting at the Bentley School of Accounting and Finance, graduating in 1925. While attending Bentley worked two summers on costs and payroll for George Killorin, Building Contractor on resident construction, Wakefield, Mass. Later engaged in public accounting and income tax work during the winter seasons and in various activities including work for the Bay State Dredging and Contracting Co., during the summers. Appointed Dredging Inspector for Commonwealth of Massachusetts, Department of Public Works, from August, 1939 to January, 1942; employed as Dredging Inspector by War Department U. S. Engineer Office, April, 1942 to January, 1943. The following three years was engaged on the accounting staff of Patterson, Teel and Dennis, Boston, Mass. At present employed by Fay, Spofford & Thorndike, Boston, Mass., as inspector, July, 1946 to October, 1946 and as estmator from October, 1946 to date. Refers to *H. L. Crocker, C. A. Farwell, W. L. Hyland, F. L. Lincoln, M. H. Mellish.*

HENRY J. PICKERSGILL, Barrington, Rhode Island. (b. September 8, 1908, East Providence, Rhode Island). B.S. in Civil Engineering, Rhode Island State College in 1932. Experience, 1932-1935, Surveyman, Rhode Island State Board of Roads; 1935-1936, Draftsman, Rhode Island State Board of Roads; 1936-1938, Draftsman, Naval Torpedo Station, Newport, R. I.; 1938-1939, Engineering Aide, U. S. Engineer Office, Providence, R. I.; 1939-1940, Junior Engineer, Fortification Division, U. S. Engineer Office, Providence, R. I.; 1940-1941, Assistant Engineer to Associate Engineer, U. S. Engineer Office, Philadelphia, Penna.; 1941-1943, Associate Engineer to Engineer, Fortification Division, U. S. Engineer Office, Providence, R. I., in responsible charge of design and engineering all fortification construction H. D. Long Island

Sound, Narragansett Bay, New Bedford; 1943 to date, Engineer (structural), Flood Control Division, U. S. Engineer Office, Providence, R. I., then Office of Division Engineer, Corps of Engineers, Boston, Mass. Presently Chief, Projects Section, Flood Control Branch, Engineering Division in charge of planning and design of flood control structures and local protective works, Connecticut River Valley, Merrimack River Valley and others. Professional registration Rhode Island and Massachusetts. Refers to *E. F. Childs, K. Eff, J. C. Dingwall, J. B. McAleer.*

RALPH L. RANKIN, Milton, Mass. (b. September 19, 1905, Boston, Mass.) Graduate of Georgia Tech. Civil Engineering. in 1925; Special Course, Boston University, Business Administration in 1927; Special Course at Mass. Institute of Technology (review) 1932, 1933, 1934. Experience, 1928-1931, Johns-Manville, Construction Engineer; 1931-1932, Penn. Tile & Brick Company. acoustical engineer; 1932-1939, own business, structural design; 1939-1943, Bethlehem Steel Company, resident engineer; 1943-1945, Bethlehem Steel Company, Hingham, Mass., assistant plant engineer; 1945, J. R. Worcester Company, construction engineer; 1945-1947, Cram & Ferguson, structural designer; 1947 to date, Fay. Spofford & Thorndike, structural engineer; Registered Professional Engineer, Massachusetts, 1947; Member. Mass. State Association of Architects, 1947; Licensed Builder. ABC, Boston, 1945. Refers to *H. L. Crocker, G. C. Douglas, F. L. Heaney, M. J. Lorente, M. H. Mellish.*

ROGER P. REIDY, Newton Highlands, Mass. (b. March 17, 1925, Newton, Mass.) Graduated from Harvard College in 1944, with A.B. degree in Civil Engineering. Experience, served as Ensign in Civil Engineer Corps, U. S. Navy from January, 1945 to July, 1946. as construction officer in the 94th, 76th, 23rd, and 103rd Naval Construction Battalions, successively on Guam, M.I.: September, 1946, returned to Harvard Graduate School of Engineering and in June, 1947, received M.S. degree. Particular field of concentration—Soil Mechanics. July, 1947, to date, with Maurice A. Reidy, Boston, Mass., as inspector and timekeeper for Raymond Concrete Pile Company on a foundation load test for proposed Jordan Marsh Store. Refers to *E. H. Bliss, H. I. Wyner, H. A. Mohr, M. J. Caddigan.*

WESLEY F. RESTALL, Warwick, R. I. (b. June 26, 1909, Revere, Mass.) Received B.S. degree in Civil Engineering from Tufts in 1932. Attended Northeastern University Law School, 1932-1933. Experience, 1924, J. A. L. Waddell, New York, N. Y., as chainman and concrete inspector; 1925-1926 and 1927, Springfield Gas Light Company, Springfield, Mass., as rodman; 1927-1928, Boston & Maine R.R., as rodman; 1930, Mass. Department of Public Works as inspector on Boston & Worcester Turnpike; 1931, Metropolitan District Commission as inspector Alewife Brook Parkway; 1933-1936, City of Springfield, Mass., Dept. of Streets and Engineering as Chief of Party; 1936-1946, U. S. Engineer Office, Providence, R. I., in various positions from computer to project engineer on design of flood control structures and during war years in charge of a unit handling airfield design; 1946 to present, Office of Division Engineer, New England Division, Boston, Mass., as Project Engineer responsible for the design of various flood control projects. Refers to *E. F. Childs, K. Eff, J. C. Dingwall, J. B. McAleer, W. W. Zapolski.*

FREDERIC M. RICE, Natick, Mass. (b. December 9, 1902, New Rochelle, New York). Graduated from Mass. Institute of Technology in June, 1925, S.B. in Civil Engineering. Experience,

September, 1925 to June, 1926, Assistant in Civil Engineering, Mass. Institute of Technology; June, 1926 to January, 1928, Survey Party Chief, Springfield Water Works; January, 1928 to April, 1929, Construction Engineer, Atmospheric Nitrogen Corporation, Hopewell, Va.; May, 1929 to September, 1931, Assistant Engineer, New-Kanawha Power Company, Hinton, W. Va.; September, 1931 to June, 1932, Graduate Student, Mass. Institute of Technology, S.M. in Civil Engineering; June to September, 1935 and March, 1936 to July, 1938, Dredging Inspector, U. S. Corps of Engineers, Boston; September, 1935 to March, 1936, Junior Engineer, U. S. Forest Service; July, 1938 to date, U. S. Corps of Engineers, New England Division, at present Chief of Projects and Reports Branch, River and Harbor Division, in charge of preparation of engineering reports on navigation projects. Refers to *J. F. Brittain, E. F. Childs, J. B. McAleer, M. H. Mellish, H. I. Wyner.*

SHOU-I TSIEN, Cambridge, Mass. (b. July 22, 1918, Shanghai, China). Graduated from the Chiao-tung University, Shanghai, in June, 1939, received B.S. degree. Major study was structure and railroad engineering. As a student engineer and then, as assistant engineer, worked for the Chinese government by joining the Engineerig Bureau of Yunnan-Burma Railroad. Work included surveying, drafting, designing and supervision of railroad construction, bridges and other structures. Was in charge of a section for about a year. Due to the fall of Burma to Japanese in 1942, the entire organization repatriated and liquidated, though almost 90% of the work had been completed at that time. Few months later was sent by the Government to this country for advanced studies. Was awarded the degree of Master of Science in C.E. on February, 1944 and Doctor of Science in C.E. on September, 1946, from Mass. Institute of Technology. Principal field of interest was soil-mechanics and structure whereas my subordinate were engineering materials and transportation engineering. During my stay in school, did some research work in soil-mechanics laboratory. After graduation worked for several months in New York City and then with Fay, Spofford & Thorndike, Boston, until present date. Put most of the time on designing the structures by adopting soil-mechanics principles. Sometimes, analyzed the field test data about soils and wrote some reports. My future career will continue along this line and possibly, shall head a national soil-mechanics laboratory in China and hope to start some research work in this field. Refers to *J. B. Babcock, M. H. Mellish, D. W. Taylor, K. Terzaghi, J. B. Wilbur.*

FRANCIS W. SMITH, Norwood, Mass. (b. November 30, 1890, New Britain, Conn.) Graduated Sheffield Scientific School, Yale University, Mechanical Engineering in 1912. Experience, 1912-1913, engineering apprentice, Westinghouse Machine Co., East Pittsburg, Pa.; 1913-1915, Assistant Commercial Engineer, West Penn. Traction & Water Power Company, Pittsburg, Pa.; 1915-1917, Assistant to Works Engineer, Remington Arms-Union Metallic Cartridge Company, Bridgeport, Conn.; 1917-1922, Assistant Engineer, American Agricultural Chemical Company, Boston and New York; 1922-1930, Plant Engineer, Industrial Engineer, Bird & Son, Inc., East Walpole, Mass.; 1930-1933, Works Manager, Holliston Mills, Inc., Norwood, Mass.; 193-1937, Plant Manager, East Braintree Finishing Company, East Braintree, Mass.; 1937-1941, self employed; 1941-1942, Mechanical Engineer, Boston Ordnance District, War Department, Boston, Mass. 1942-1946, General Manager, Town of Norwood, Norwood, Mass.; at present, Senior Engineer (Department Engineer) Fay, Spofford &

Thorndike, Boston, Mass. Refers to *C. A. Farwell, R. W. Horne, M. H. Mellish, C. M. Spofford.*

JOHN S. TEWKSBURY, Watertown, Mass. (b. February 25, 1925, Boston, Mass.) July, 1943 to November, 1944, attended Duke University and completed 2 years of the Civil Engineering curriculum. November, 1944 to February, 1946, continued Civil Engineering studies at the University of Michigan where I received B.S. degree. July, 1946 to date, with Fay, Spofford & Thorndike, as draftsman in the Sanitary Department. Refers to *H. L. Crocker, F. L. Heaney, L. M. Gentleman, M. H. Mellish.*

FRED E. TIBBETS, JR., Somerville, Mass. (b. January 25, 1909, Somerville, Mass.) 1929, Hebron Academy, Hebron, Maine, Scientific Course; 1938, Lowell Institute, Cambridge, Mass., 2 yr. structural course; 1939, Lowell Institute, refresher courses, building estimating and hydraulics; 1940, Tufts College, defense courses, concrete test and inspection and reinforced concrete design; University State Extension Course (refresher), structural design to present date. Experience, 1930, 1932 through 1934; 1936-1940, City of Somerville Engineering Department, Grade I and II, civil engineer field and office; 1931-1932; 1934-1936, U. S. Engineer Department, Cape Cod Canal, Mass., engineer field and office, Cape Cod Canal widening and deepening project; 1936 (3 mo.), with Fay, Spofford & Thorndike, as inspector of borings for Cape Cod Canal Bridges; 1941, M. A. Reidy, Consulting Engineer, as co-ordinator of underground utilities for So. Weymouth Lighter than Air Base; 1942 (4 mo.), Fay, Spofford & Thorndike, civil engineer, designer on drainage for So. Boston Dry Dock; 1942-1943, M. A. Reidy, consulting engineer, Brown & Matthews, Contractors and Engineers, Providence, R. I., civil engineer on design-layout and supervision of underground utilities and dredging; 1934-1945, Walsh-Kaiser Company, Inc., Providence, R. I., piping engineer, marine design Walsh-Kaiser Ship Yard, engineer responsible for fabrication, installation and function of various large piping systems in new boats, 15,000 ton class; 1945-1946, M. A. Reidy, Boston, chief field engineer on repairs to underground utilities at Old Harbor Village Housing Project; 1946 (9 mo.), C. J. Maney Co., Somerville, Mass., chief field engineer on demounting, transporting, erecting and altering temporary Veterans Program under F.P.H.A.; 1947 to date, Fay, Spofford & Thorndike, engineer in Sanitary Department on design and layout of underground utilities, Alaska Project. Refers to *G. C. Douglas, C. A. Farwell, F. J. Heaney, W. L. Hyland, M. H. Mellish.*

FREDERICK T. WEED, Lynn, Mass. (b. September 17, 1893, Boston, Mass.) Graduated Lynn Eng. High School in 1912; from Northeastern University Evening Polytechnic School, Structural Engineering in June, 1924. Experience, 1912-1917, employed Eastman & Bradford, C.E.; 1917, U. S. Army Eng.; 1918, enlisted Signal Corps, U.S.A. Served at University of Vermont and with 211th Field Sig. Battalion, U.S.A., discharged in 1919; 1919-1922, employed Eastman & Bradford, C.E.; January, 1923 to date, member of firm of Bradford & Weed, Lynn, Mass., Civil Engineers and Surveyors. Commissioned 2nd Lt., C.E., ORC, December, 1926; 2nd Lt., C.E.N.G.U.S., April, 1929, assigned to 101st Engineers, Mass. N.G. Battalion Adj. Demolition Officer; Captain, C.E.N.G.U.S., assigned as Commanding Officer Co. D, 101 Eng., 1935 through 1940. Called into active military service with 101 Eng., 1941. Served as Company C.O., Asst. Div. Eng. 26th Div. Plans and Training Officer, 101 Eng., 1942. Transferred to Army Air Forces with Staff

Headquarters North Atlantic Division A.T.C. as Intelligence Officer, Map Reproduction Officer; S-2 and Briefing Officer and as Division Reproduction Officer. Service as Captain, Major and Lieut. Colonel AC. Discharged April, 1946, as Lt. Col. Corps Eng. Reserve. 1935-1940, Instructor in Map Reading, Massachusetts Military Academy, Mass. N.G., September, 1946. Commissioned and qualified as Lieut. Colonel, Corps of Engineer, N.G.U.S., assigned as State Engineer Officer on the Staff of the Adjutant General, Massachusetts National Guard, to date. Resumed active practice of engineering 1946, with firm of Bradford & Weed. General municipal engineering practice. Registered with the Commonwealth of Masachusetts as Professional Engineer, Certificate No. 1745 and Registered Land Surveyor, Certificate No. 1735. Refers to *C. O. Baird, C. B. Humphrey, W. E. Nightingale, W. B. Hilton.*

WINTHROP A. WELLS, Arlington, Mass. (b. July 29, 1915, Bristol, New Hampshire). Graduate of Wesleyan University, Middleton, Conn., June, 1937, B.A. in Math., Mass. Institute of Technology. Summer, school, 1941, Soil Mechanics Course. Eng. Def. Training Course, Clarkson Tech., Potsdam, N. Y. Certificate in Structures. Experience, November, 1938 to February, 1939, Computer in Soils Lab., Corps of Engineers, Concord, N. H.; May, 1939, rodman for State Highway Department, New Hampshire; July, 1939 to November, 1940, Computer and Laboratory Assistant, Soils Lab., Corps of Engineers, Concord, N. H.; December, 1940 to March, 1942, setting up and running soils laboratory. Making stability and stress analyses of canal slopes, dikes, and earth abutments, St. Lawrence River Seaway Survey, Corps of Engineers, Massena, N. Y.; March, 1942 to June, 1942, Assistant Office Engineer and Inspector on 230 KV power line construction, Corps of Engineers, Canton, N. Y.; July, 1942 to November, 1942, Sub-party chief on 40-man survey party on transmission on line survey, Corps of Engineers, Cooperstown, N. Y.; November, 1942 to February, 1944, Assistant to Engineer in Charge of Soils and Geology Section, Boston District, Corps of Engineers; February, 1944 to November, 1946, Assistant to Engineer in Charge of Frost Investigation Section, Boston District, Corps of Engineers; November, 1946 to October, 1947, Engineer in Charge of Frost Investigation Section, New England Division, Corps of Engineers, directing research of design of airfield pavements in areas where detrimental frost action occurs. Refers to *E. F. Childs, J. B. McAleer, H. F. Perdikis, D. W. Taylor.*

HOWARD J. WILLIAMS, Braintree, Mass. (b. April 28, 1895, Kingston, Canada). Experience, April to November, 1917, Field Engineer on Construction Work, Cedar Rapids Power Company, Cedars, Quebec; November, 1917 to September, 1919, Field Engineer on Construction Work, Hydro Electric Power Commission of Ontario on Chippewa-Queenstown Power Development, Niagara Falls, Ontario; October, 1919 to June, 1920, graduate studies and research at Massachusetts Institute of Technology for Master of Science Degree; June, 1920 to February, 1923, Assistant Engineer, Maine Water Power Commission. Computations and studies of river discharge, evaporation, storage and power sites; office and field investigations of developed and undeveloped power resources of the State; March to April, 1923, Assistant Engineer with John F. Vaughan, Consulting Engineer, Boston, Massachusetts, on studies of stream flow and storage; April, 1923 to July, 1926, Assistant Engineer, Designing Division, Water Supply Board, Providence, Rhode Island. Hydraulic and structural computations and designs for Scituate Dam, Hydro-Electric Plant and Aqueduct. Studies for valuations of condemned mills and water

powers and co-ordinating the work of lawyers and expert witnesses in preparation of testimony for hearings on $6,000,000 suit. July, 1926 to July, 1947, Senior Engineer with Fay, Spofford and Thorndike, Consulting Engineers, Boston, Mass. Principal hydraulic and structural designer on bridges, buildings, dams, reservoirs and waterfront structures. Preparation of plans, specifications and reports. July, 1947 to date, partner, Fay, Spofford and Thorndike, Consulting Engineers, Boston, Mass. Refers to *J. F. Brittain, C. A. Farwell, L. M. Gentleman, F. L. Lincoln, J. B. Wilbur.*

Transfer from Grade of Junior

JOSEPH C. LAWLER, Lynn, Mass. (b. May 3, 1920, Lynn, Mass.) Graduated from Northeastern University in 1943, B.S. degree in Civil Engineering; M.S. in Sanitary Engineering, Harvard University in 1947. Experience, April, 1940 to August, 1940, rodman and transitman with W. S. Crocker, Civil Engineer, Boston, Mass.; November, 1940 to November, 1943, draftsman and junior engineer with Samuel M. Ellsworth, Consulting Engineer; February, 1944, to June, 1946, U.S.N.R., Lt. (jg) Civil Engineering Corps. Officer in Charge of Construction Battalion Maintenance Unit of 270 men on Guam. Officer in Charge of C.B. M.U. of 400 men on Eniwetok, Public Works Officer of Eniwetok Atoll on Crossroads project. At present Assistant Engineer with Camp, Dresser and McKee, Consulting Engineers, Boston, Mass. Refers to *T. R. Camp, H. G. Dresser, E. A. Gramstorff, J. E. McKee, H. M. Turner.*

LOUIS P. VUONA, Worcester, Mass. (b. August 14, 1919, Worcester, Mass.) Graduated from Northeastern University in June, 1947, B.S. degree in Civil Engineering. Experience, July, 1943 to October, 1946, served with U. S. Marine Corps Reserve, rank of 1st Lieutenant, designated engineer officer and assigned engineering dtuies. June, 1947 to date, employed by Vuona Bros., Contractors, Worcester, Mass., duties consist of estimating, planning and supervision of sub-contract work, primarily excavation. Refers to *C. O. Baird, E. A. Gramstorff, G. W. Hankinson, E. L. Spencer.*

Transfer from Grade of Student

IRVING T. BERKLAND, Norwood, Mass. (b. August 14, 1920, Norwood, Mass.) Graduated from Northeastern University in April, 1943, receiving B.S. degree in Civil Engineering. Cooperative work consisted of working for the N. Y., N. H. & H. Railroad in the Division Engineers Office in Providence, R. I. Experience obtained includes surveying, from chainman to chief-of-party, drafting, tracing, making up authorities for expenditure, estimating and track layout. With S. M. Ellsworth and Howard M. Turner, Consulting Engineers, for a period of five months. Work consisted of drafting and tracing elements of sewage treatment plants, calculations involving plant depreciation and hydrographic research calculations. Experience, April, 1943 to August, 1944, with Carnegie-Illinois Steel Corp. in Clairton, Pa. Work consisted of observation of steel making process for quality control purposes. August, 1944, entered the U. S. Navy (communication work) and was discharged Lt. (jg) in August, 1946. After an 8 months' try at bookbinding (estimating, costwork, labor grievances) joined the staff of Northeastern University on August 4, 1947. Work consists of placing Civil and Industrial Engineering students in cooperative work positions. Refers to *C. O. Baird, H. G. Dresser, E. A. Gramstorff, G. W. Hankinson, W. E. Nightingale.*

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DEATHS

Charles A. Mixer, October 24, 1947
Arthur G. Robbins, October 26, 1947
Harry M. Steward, January 3, 1948

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