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Volume XXXV OCTOBER, 1948 Number 4

VERTICAL EASEMENT CURVES

By Charles O. Baird, Member*

(Presented at a meeting of the Surveying and Mapping Section of the Boston Society of Civil Engineers, held on April 30, 1947.)

The intersection of two different grades on a highway, railroad or an airport runway forms a vertex. To provide a smooth transition from one grade to another a vertical easement curve is commonly employed. When the vertex is above the vertical easement curve, the curve is known as a summit and when the vertex is below, the curve is known as a sag.

Horizontal curves are commonly circular arcs which may be readily laid out by means of deflection angles and chord distances. Vertical transition curves do not lend themselves to this method of layout but are generally laid out by means of differential leveling. While the necessary computations for satisfactory laying out vertical easement curves are relatively simple, the writer feels that a review of the various principles involved plus a discussion of sight distances and the effects of acceleration is of interest. In addition, the writer has prepared several graphs and tables to facilitate the computation of vertical curves.

Elevations for laying out vertical easement curves are generally obtained by computing vertical offsets from one or both of the intersecting grade lines. A portion of a circular arc is shown in Figure 1. The center of the circular curve is at the intersection of X_1 and Y axes; while points are located on this curve by distances measured along the X axis and offsets from the X axis. When the origin is at

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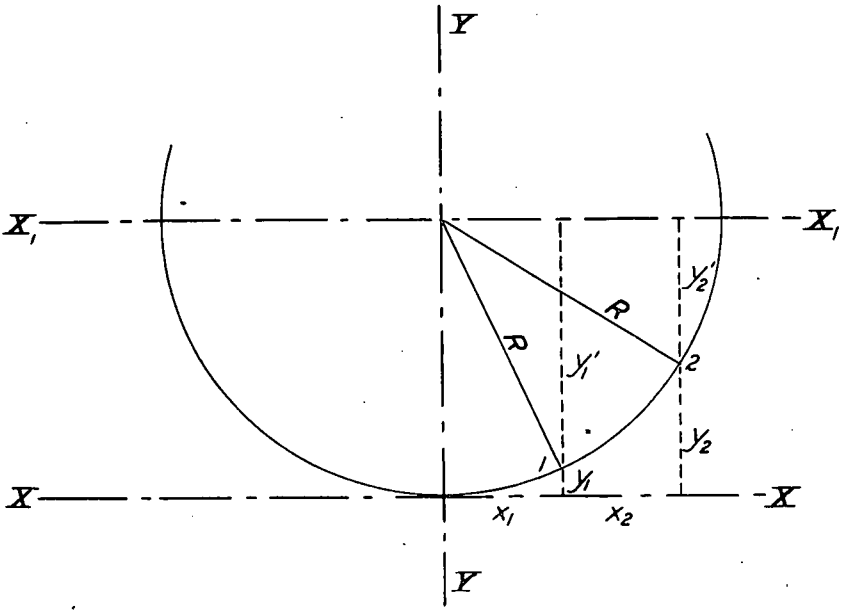


CIRCULAR CURVE
FIGURE 1

the intersection of axes X_1 and Y, the circular arc may be expressed by the equation of the circle:

(1) $$x^2+y^2 = R^2$$

If the origin be moved to the intersection of the axes X and Y, the two points on the circular arc may be located by $(x_1)(y_1)$ and $(x_2)(y_2)$ respectively. The symbols $y^1{}_1$ and $y^1{}_2$ represent the y value of the two points for equation (1) and in terms of R and x are equal to:

$$(y^1{}_1)^2 = R^2 - x^2{}_1 \text{ and } (y^1{}_2)^2 = R^2 - x^2{}_2$$

from which:

$$y^1{}_1 = \sqrt{R^2 - x^2{}_1} \text{ and } y^1{}_2 = \sqrt{R^2 - x^2{}_2}$$

The offsets from the axis of X to points 1 and 2, expressed in terms of R and y^1 become:

$$y_1 = R - y^1{}_1 \quad \text{and} \quad y_2 = R - y^1{}_2$$

Substituting previous value of y^1 the offsets may be expressed as:

$$y_1 = R - \sqrt{R^2 - x^2_1} \quad \text{and} \quad y_2 = R - \sqrt{R^2 - x^2_2}$$

Then the ratio $\frac{y_2}{y_1}$ is:

$$\frac{y_2}{y_1} = \frac{R - \sqrt{R^2 - x^2_2}}{R - \sqrt{R^2 - x^2_1}}$$

or:

$$(2) \qquad y_2 = \frac{y_1[\,R - \sqrt{R^2 - x^2_2}\,]}{[\,R - \sqrt{R^2 - x^2_1}\,]}$$

This equation (2) is cumbersome to use, thereby defeating the purpose of an easy and a simple method of calculating perpendicular offsets. From the familiar slope correction problem of taping, we have the approximate formula for slope correction equal to $\frac{h^2}{2S}$. Applying this expression to the nomenclature used in figure 1, the values y^1_1 and y^1_2 become

$$y^1_1 = R - \frac{x^2_1}{2R} \quad \text{and} \quad y^1_2 = R - \frac{x^2_2}{2R}$$

but as above:

$$y_1 = R - y^1_1 \quad \text{and} \quad y_2 = R - y^1_2$$

and by substitution:

$$y_1 = \frac{x^2_1}{2R} \quad \text{and} \quad y_2 = \frac{x^2_2}{2R}$$

Then the ratio $\frac{y_2}{y_1}$ is:

$$\frac{y_2}{y_1} = \frac{x^2_2}{x^2_1}$$

From which:

$$(3) \qquad y_2 = y_1 \frac{x^2_2}{x^2_1} \qquad \text{(approximately)}$$

This approximate equation (3) is easy to apply and in most cases can be calculated upon the slide rule. This formula may be used for calculations to the nearest $\frac{1}{100}$ of a foot, when the tangent of the angle is equal to or smaller than 0.140.

The equation of the square parabola is:

$$x^2 = 4py$$

In figure 2, the X axis is tangent to the parabola at the intersection of the X and Y axes. Two offset values y_1 and y_2 are shown and the expression for these two points become respectively:

$$x^2_1 = 4py_1 \qquad \text{and} \qquad x^2_2 = 4py_2$$

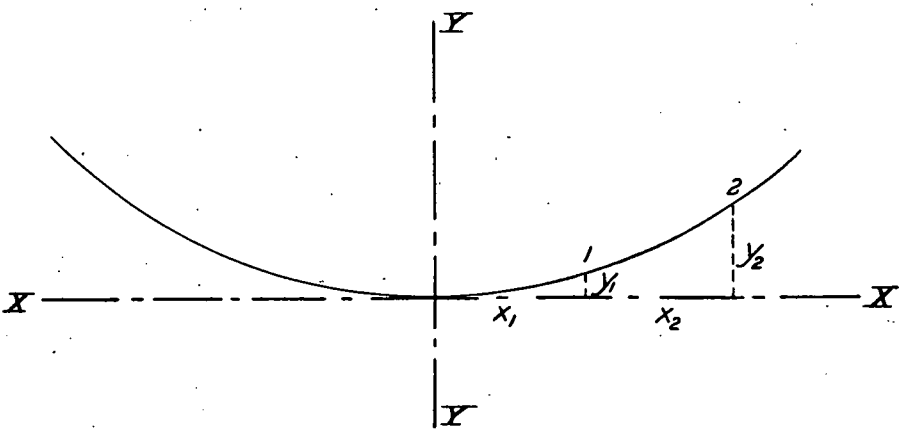
and when one is divided by the other:

$$\frac{x^2_1}{x^2_2} = \frac{y_1}{y_2}$$

whence:

$$y_2 = y_1 \frac{x^2_2}{x^2_1} \tag{4}$$

Equation (4) may be stated as follows: the offsets from the tangent to the parabolic curve vary directly as the squares of their tangent distances. It will be noted that equations (3) and (4) are identical, and for practical considerations a circular curve or a parabolic curve are one and the same thing.



PARABOLIC CURVE
FIGURE 2

From the vertical curve shown in figure 3, it can be demonstrated that the total offset, a_n is:

$$a_n = \frac{g_1 n}{2} - \frac{g_2 n}{2}$$

or:

(5) $$a_n = \frac{n}{2}(g_1 - g_2)$$

The quantity $(g_1 - g_2)$ is frequently called the algebraic difference in grades, and n is the length of the vertical curve expressed in stations. The total offset, a_n may be computed as an offset for a parabolic curve by equation number 4:

(6) $$a_n = a_1 \left(\frac{n}{1}\right)^2$$

The offset a_1 is the offset from the tangent one (1) station from the point of tangency. When equations 5 and 6 are compared:

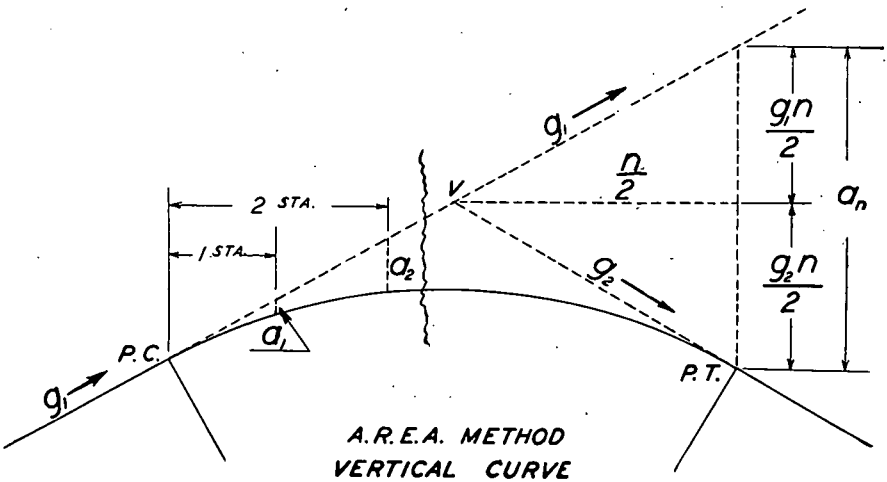
$$a_1 n^2 = \frac{n}{2}(g_1 - g_2)$$

from which:

(7) $$a_1 = \frac{(g_1 - g_2)}{2n}$$

or:

(8) $$n = \frac{(g_1 - g_2)}{2a_1}$$



A.R.E.A. METHOD
VERTICAL CURVE

FIGURE 3

Equation 7 may be written in logarithmic form as:

$$\text{Log. } a_1 = \text{Log. } (g_1 - g_2) - \text{Log. } 2n$$

The log. a_1 is plotted on the abscissa, while the log. $(g_1 - g_2)$ is plotted on the ordinate, and the results are shown in figure 4. The results of equation 7 are, also, shown in Table I for lengths of curve varying by even numbered stations from 2 to 20 stations, and for values of $(g_1 - g_2)$ from 0.01 to 20.00.

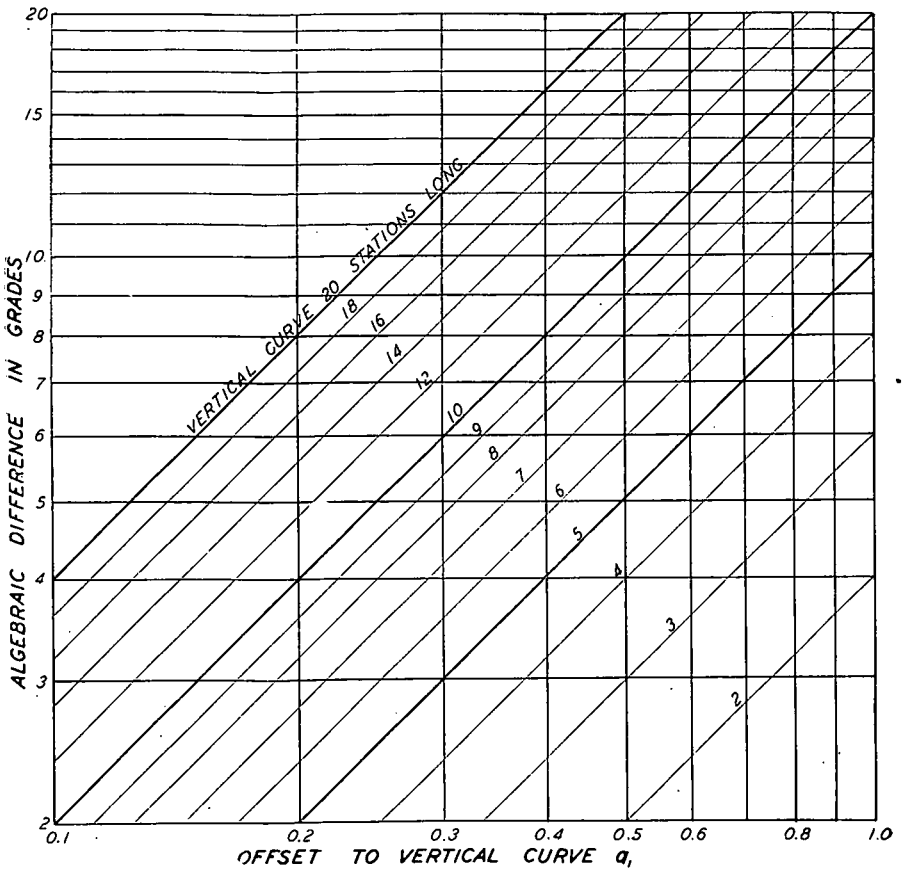


FIGURE 4

VALUES OF a_1

TABLE I

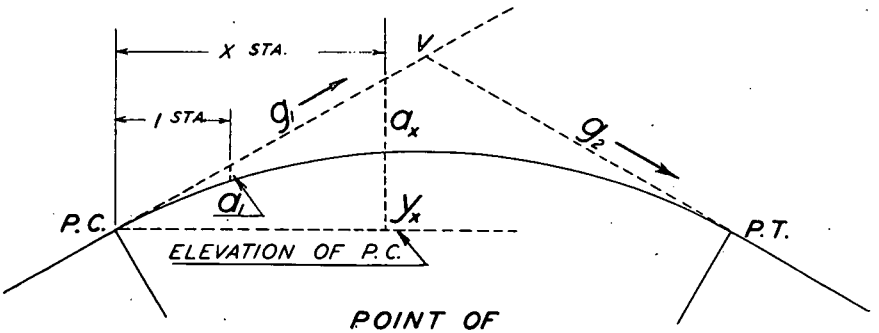
ALGEBRAIC DIFFERENCE IN GRADES	LENGTH OF CURVE 2	4	6	8	10	12	14	16	18	20
0.01	0.00250	0.00125	0.00083	0.00063	0.00050	0.00042	0.00036	0.00031	0.00028	0.00025
0.02	0.00500	0.00250	0.00167	0.00125	0.00100	0.00083	0.00071	0.00062	0.00056	0.00050
0.03	0.00750	0.00375	0.00250	0.00188	0.00150	0.00125	0.00107	0.00094	0.00083	0.00075
0.04	0.01000	0.00500	0.00333	0.00250	0.00200	0.00167	0.00143	0.00125	0.00111	0.00100
0.05	0.01250	0.00625	0.00417	0.00313	0.00250	0.00208	0.00179	0.00156	0.00139	0.00125
0.06	0.01500	0.00750	0.00500	0.00375	0.00300	0.00250	0.00214	0.00188	0.00167	0.00150
0.07	0.01750	0.00875	0.00583	0.00438	0.00350	0.00292	0.00250	0.00219	0.00194	0.00175
0.08	0.02000	0.01000	0.00667	0.00500	0.00400	0.00333	0.00286	0.00250	0.00222	0.00200
0.09	0.02250	0.01125	0.00750	0.00563	0.00450	0.00375	0.00321	0.00281	0.00250	0.00225
0.10	0.02500	0.01250	0.00833	0.00625	0.00500	0.00417	0.00357	0.00312	0.00278	0.00250
0.20	0.05000	0.02500	0.01667	0.01250	0.01000	0.00833	0.00714	0.00625	0.00556	0.00500
0.30	0.07500	0.03750	0.02500	0.01875	0.01500	0.01250	0.01071	0.00938	0.00833	0.00750
0.40	0.10000	0.05000	0.03333	0.02500	0.02000	0.01667	0.01429	0.01250	0.01111	0.01000
0.50	0.12500	0.06250	0.04167	0.03125	0.02500	0.02083	0.01786	0.01562	0.01389	0.01250
0.60	0.15000	0.07500	0.05000	0.03750	0.03000	0.02500	0.02143	0.01875	0.01667	0.01500
0.70	0.17500	0.08750	0.05833	0.04375	0.03500	0.02917	0.02500	0.02188	0.01944	0.01750
0.80	0.20000	0.10000	0.06667	0.05000	0.04000	0.03333	0.02857	0.02500	0.02222	0.02000
0.90	0.22500	0.11250	0.07500	0.05625	0.04500	0.03750	0.03214	0.02812	0.02500	0.02250
1.00	0.25000	0.12500	0.08333	0.06250	0.05000	0.04167	0.03571	0.03125	0.02778	0.02500
2.00	0.50000	0.25000	0.16667	0.12500	0.10000	0.08333	0.07143	0.06250	0.05556	0.05000
3.00	0.75000	0.37500	0.25000	0.18750	0.15000	0.12500	0.10714	0.09375	0.08333	0.07500
4.00	1.00000	0.50000	0.33333	0.25000	0.20000	0.16667	0.14286	0.12500	0.11111	0.10000
5.00	1.25000	0.62500	0.41667	0.31250	0.25000	0.20833	0.17857	0.15625	0.13889	0.12500
6.00	1.50000	0.75000	0.50000	0.37500	0.30000	0.25000	0.21429	0.18750	0.16667	0.15000
7.00	1.75000	0.87500	0.58333	0.43750	0.35000	0.29167	0.25000	0.21875	0.19444	0.17500
8.00	2.00000	1.00000	0.66667	0.50000	0.40000	0.33333	0.28572	0.25000	0.22222	0.20000
9.00	2.25000	1.12500	0.75000	0.56250	0.45000	0.37500	0.32143	0.28125	0.25000	0.22500
10.00	2.50000	1.25000	0.83333	0.62500	0.50000	0.41667	0.35715	0.31250	0.27778	0.25000
11.00	2.75000	1.37500	0.91667	0.68750	0.55000	0.45833	0.39286	0.34375	0.30556	0.27500
12.00	3.00000	1.50000	1.00000	0.75000	0.60000	0.50000	0.42857	0.37500	0.33333	0.30000
13.00	3.25000	1.62500	1.08333	0.81250	0.65000	0.54167	0.46429	0.40625	0.36111	0.32500
14.00	3.50000	1.75000	1.16667	0.87500	0.70000	0.58333	0.50000	0.43750	0.38889	0.35000
15.00	3.75000	1.87500	1.25000	0.93750	0.75000	0.62500	0.53572	0.46875	0.41667	0.37500
16.00	4.00000	2.00000	1.33333	1.00000	0.80000	0.66667	0.57143	0.50000	0.44444	0.40000
17.00	4.25000	2.12500	1.41667	1.06250	0.85000	0.70833	0.60715	0.53125	0.47222	0.42500
18.00	4.50000	2.25000	1.50000	1.12500	0.90000	0.75000	0.64286	0.56250	0.50000	0.45000
19.00	4.75000	2.37500	1.58333	1.18750	0.95000	0.79167	0.67858	0.59375	0.52778	0.47500
20.00	5.00000	2.50000	1.66667	1.25000	1.00000	0.83333	0.71429	0.62500	0.55556	0.50000

The American Railroad Engineering Association (hereafter abbreviated to A.R.E.A.) method of solution for vertical curves is developed by referring to figure 3 as:

$$\text{Elev. Sta. } 1 = \text{Elev. Sta. } 0 + (g_1 - a_1)$$
$$\text{Elev. Sta. } 2 = \text{Elev. Sta. } 1 + (g_1 - 3a_1)$$
$$\text{Elev. Sta. } 3 = \text{Elev. Sta. } 2 + (g_1 - 5a_1)$$

and at any station:

(9) $$\text{Elev. Sta. } x = \text{Elev. P. C.} + g_1 \cdot x - a_x$$



POINT OF
MAXIMUM OR MINIMUM ELEVATION
VERTICAL CURVE
FIGURE 5

Figure 5 shows a general sketch of a vertical curve adapted to indicate the elevation of any point upon the curve such as station x. It will be noted that the distance y_x is the variable to be added to the elevation of the P. C. in order to find the elevation of station x. The value of y_x may be expressed as:

(10) $$y_x = g_1 \cdot x - a_x$$

but:

$$a_x = a_1 \cdot x^2$$

From equation (7):

$$a_1 = \frac{(g_1 - g_2)}{2 \cdot n}$$

So equation (10) may be written as:

(11) $$y_x = g_1 \cdot x - \frac{(g_1 - g_2)x^2}{2 \cdot n}$$

If the value of y_x (equation 11) is differentiated with respect to x and set equal to zero we have:

$$d\left(g_1 \cdot x - \frac{(g_1 - g_2)x^2}{2 \cdot n}\right) = 0 = g_1 - \frac{x(g_1 - g_2)}{n}$$

From which:

$$x = \frac{g_1 \cdot n}{(g_1 - g_2)} \tag{12}$$

Therefore, equation 12 locates the point of maximum or minimum elevation on a vertical curve, and if this station be inserted in equation 9, the elevation of this point can be readily computed. If equation 12 be written in logarithmic form it becomes:

$$\text{Log. } x = \text{Log. } g_1 + \text{Log. } n - \text{Log. } (g_1 - g_2) \tag{13}$$

Equation 13 is plotted with the Log. $(g_1 - g_2)$ as the abscissa; with the Log. g_1 as the ordinate; when the value of n is equal to 10 stations. The results of this plot are shown in figure 6. In order to find the station x; when n, the length, is any other value other than 10; it is only necessary to multiply the value found in figure 6 by its proportionality factor. These factors, or multipliers, are set forth in Table II as a matter of convenience.

TABLE II

Length of Curve in Stations	Multiplier
4	0.4
6	0.6
8	0.8
10	1.0
12	1.2
14	1.4
16	1.6
18	1.8
20	2.0

The length of a vertical curve may be determined by several methods. In this prospectus, the length of the curve will be studied by the acceleration of the vehicle, the sight distance for summits, and the sight distance for sags. The tangent distance for an ordinary circular curve may be expressed as:

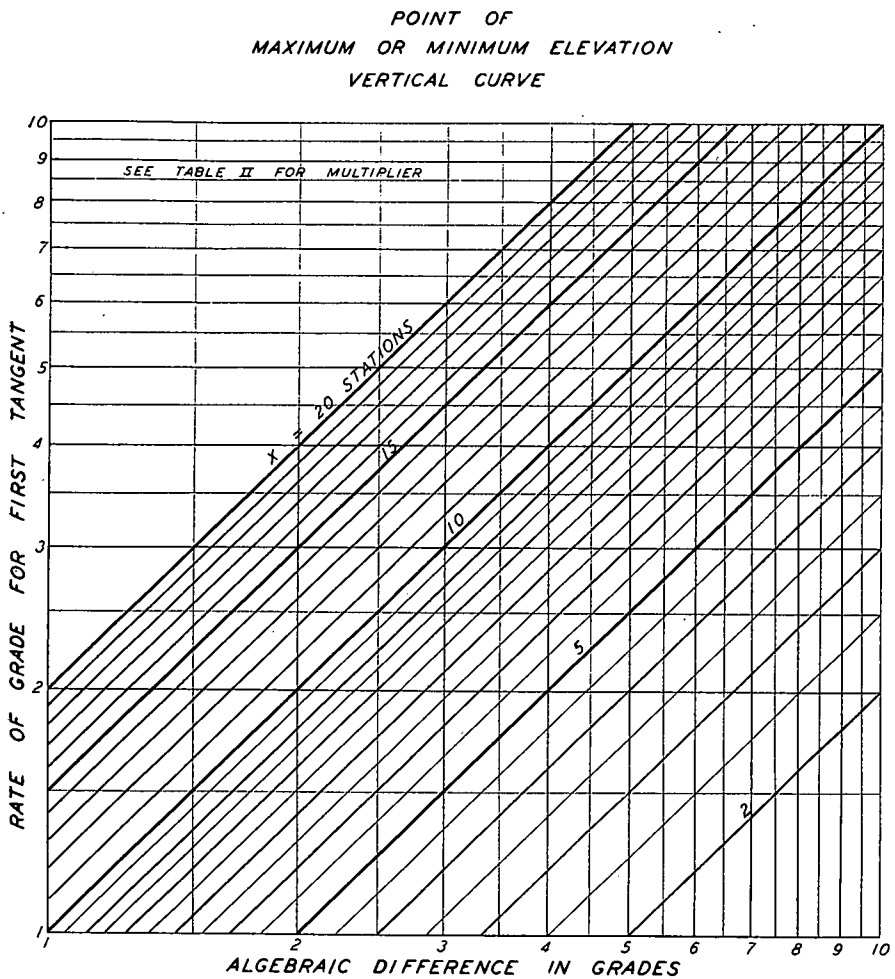


FIGURE 6

(14) $$T = R\tan\tfrac{1}{2}I$$

The tangent distance for a vertical curve may be approximated as $\frac{n \times 100}{2}$. The multiplier 100 is necessary to convert the length of

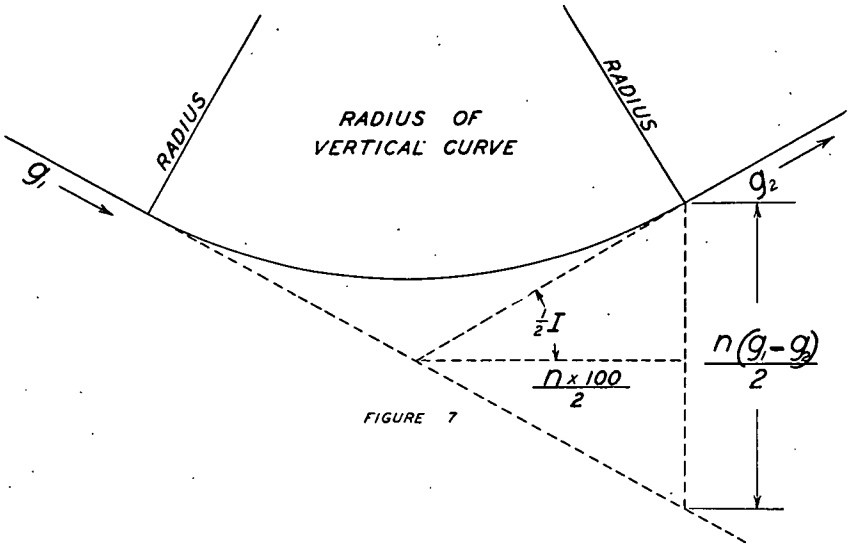


FIGURE 7

curve from stations to feet. As indicated in figure 7, the tan½I may be expressed as:

$$\text{(15)} \qquad \text{Tan}\tfrac{1}{2}\text{I} = \frac{\dfrac{n(g_1-g_2)}{4}}{\dfrac{n \times 100}{2}} = \frac{(g_1-g_2)}{200}$$

As can be shown from figure 7, the approximate tangent distance is equal to:

$$\text{(16)} \qquad T = \frac{100 \times n}{2} \qquad \text{(approximately)}$$

If equations 15 and 16 are inserted in equation 12; and then the resulting equation is solved in terms of R; R can then be approximately stated as:

$$\text{(17)} \qquad R = \frac{(100)^2 n}{(g_1-g_2)} \qquad \text{(approximately)}$$

This approximate value of R is reliable and gives values for most normal cases well within the limits of ordinary good construction work. This approximate value of R, equation 17, makes a simple method, rapidly computed upon the slide rule, available for plotting and for

other general data to follow. This equation 17 may be written in logarithmic form as:

(18) $$\text{Log. } R = 2 \text{ Log. } 100 + \text{Log. } n - \text{Log. } (g_1 - g_2)$$

The results of equation 18 are plotted and shown in figure 8. The Log. of the radius in feet is plotted on the abscissa when the Log. of the algebraic difference in grades is plotted on the ordinate.

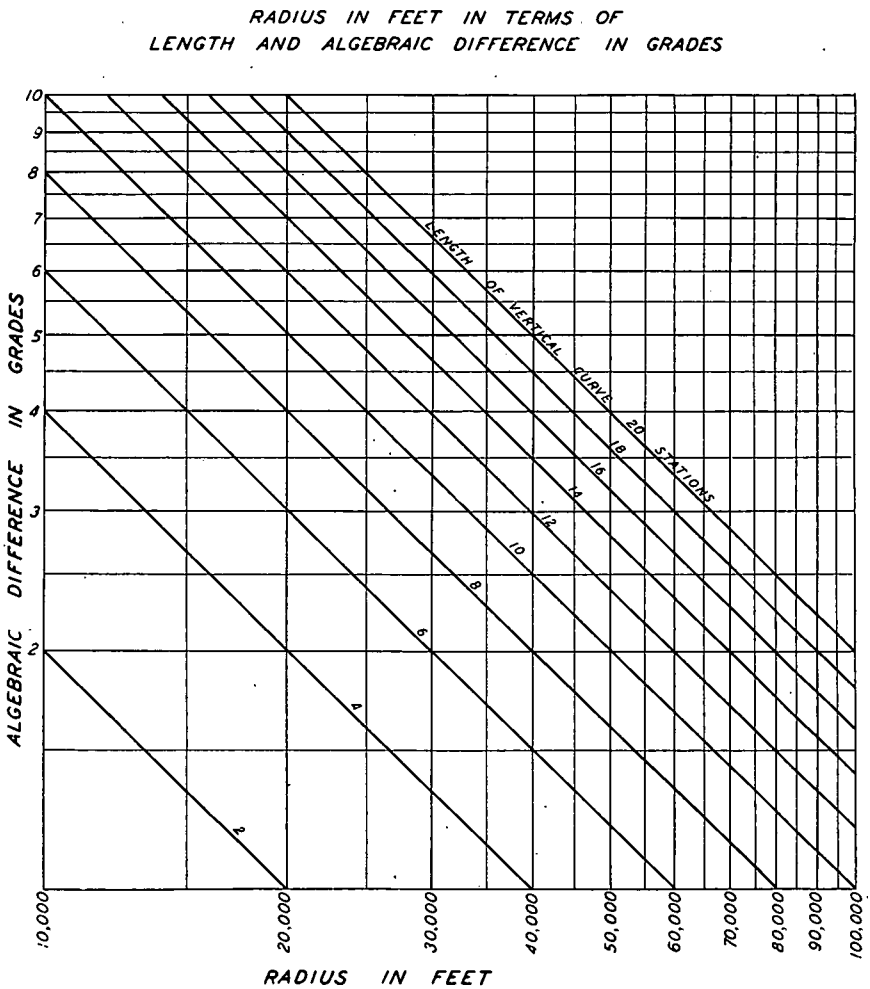


FIGURE 8

If the vertical curve is to be a true transition from one tangent to another tangent, then this transition should be accomplished without the occupant of the vehicle being aware of a change of direction. Assuming a constant velocity for forward motion then the change in tangential velocity is zero and the tangential acceleration, also, equals zero. The change in direction due to the vertical curve then will produce normal acceleration only and normal acceleration may be expressed by one of the following equations:

$$a_n = \frac{V^2}{R} \text{ or } w^2R \text{ or } Vw$$

As most engineers express velocities in miles per hour and the velocity in the normal acceleration equation is in feet per second, a conversion factor is needed to convert M.P.H. to feet per second and is:

$$1.465 \text{ M.P.H.} = \text{feet per second}$$

The normal acceleration, with velocity expressed in terms of M.P.H. then may be expressed as:

$$a_n = \frac{(1.465V)^2}{R} \text{ or } \frac{2.15V^2}{R} \tag{19}$$

What acceleration causes a passenger to become uncomfortable? There seems to be no physiological data available on the minimum values of acceleration detected by the average human being. Many English engineers have assumed a normal acceleration of 1.0 foot per second per second, while for similar problems American engineers have assumed 2.0 feet per second per second. The author has assumed 2.0 feet per second per second in all discussions that follow. Substituting the approximate value of R from equation 17 in equation 19 and solving for the length n we have:

$$n = \frac{2.15(g_1 - g_2)V^2}{(100)^2 a_n} \tag{20}$$

whence by substituting 2.0′/s/s for a and evaluating the constants becomes:

$$n = 0.0001075(g_1 - g_2)V^2 \tag{21}$$

Equation 21 may, also, be expressed in logarithmic form as:

$$\text{Log. } n = \text{Log. } (0.0001075) + \text{Log. } (g_1 - g_2) + 2 \text{ Log. } V \tag{22}$$

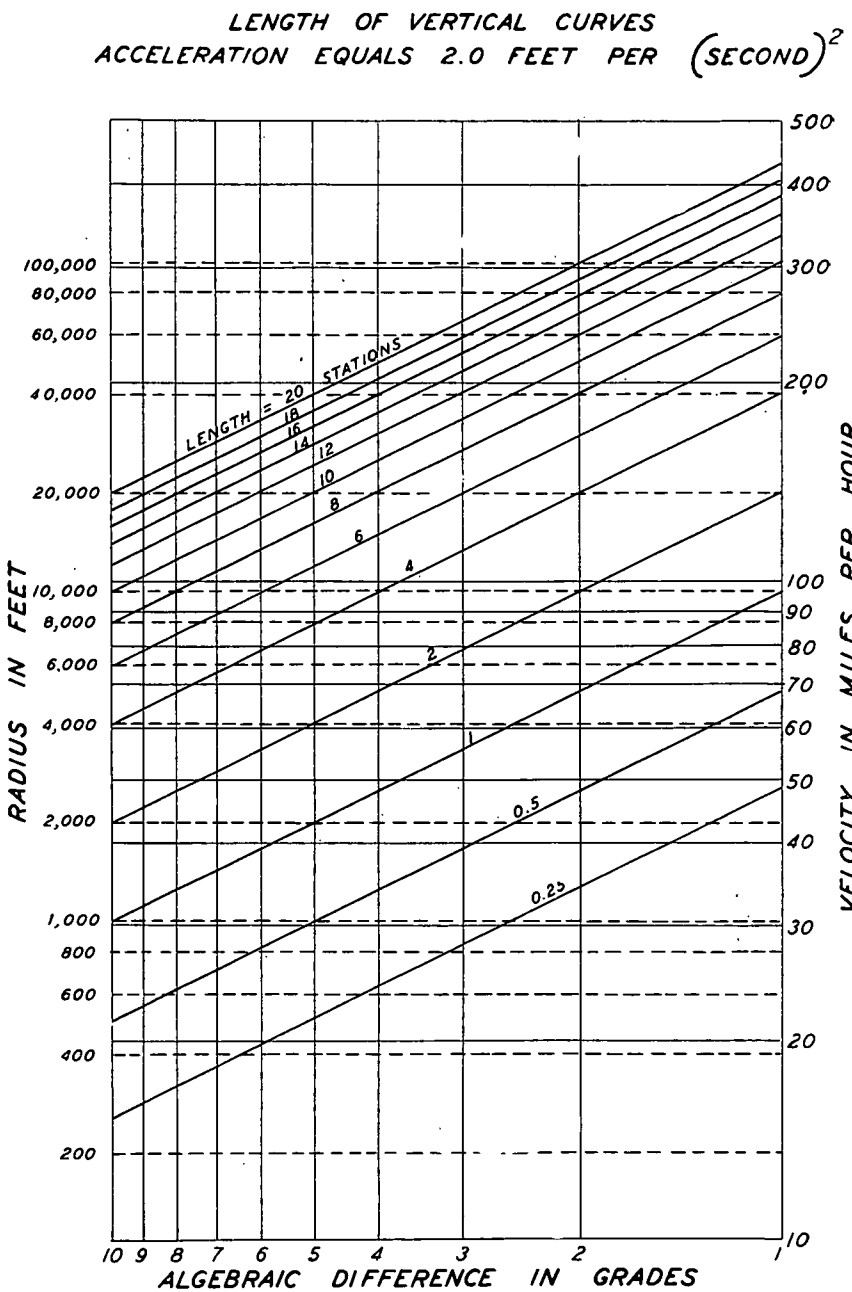

LENGTH OF VERTICAL CURVES
ACCELERATION EQUALS 2.0 FEET PER (SECOND)^2
RADIUS IN FEET
100,000
80,000
60,000
40,000
20,000
10,000
8,000
6,000
4,000
2,000
1,000
800
600
400
200
VELOCITY IN MILES PER HOUR
500
400
300
200
100
90
80
70
60
50
40
30
20
10
LENGTH = 20 STATIONS
18
16
14
12
10
8
6
4
2
1
0.5
0.25
10 9 8 7 6 5 4 3 2 1
ALGEBRAIC DIFFERENCE IN GRADES

FIGURE 8A

Figure 8A is a plot of the results of equation 22, with the algebraic difference in grades plotted on the abscissa and the velocity in miles per hour plotted on the ordinate. (Please note that this ordinate is plotted along the right hand edge of the graph.) In order to better correlate the radii data, shown in figure 8, the radii in feet are plotted upon the ordinate to the left of the sheet and the short horizontal dash lines are for the various values of R.

The sight distance on a vertical curve may have several different definitions, depending upon whether it is a summit, a sag, or a multilane highway with restricted one way traffic.

Sight Distances on Summits. For purposes of this paper, the sight distance will be defined and used in development of theory as: the linear distance from eye of one operator to the eye of a second operator driving toward each other in opposing lanes. The height of the operator's eye above the pavement is called the eye distance and will be represented by the symbol e. This eye distance may vary from 3.5 ft. to 6.0 ft. or more, however, in all cases where numerical values are used in this paper, including graphs of sight distances in terms of length of curve, 5.0 ft. has been used. The sight distance used on summits for some free way designs has been defined as the distance from the eye of the operator to a small object upon the pavement or a pavement failure. If this second definition is used, multiply this second sight distance by 2 and use the data subsequently developed in determining the length of the vertical curve by the former definition of sight distance.

Case I. Length of vertical curve when the length (h) is greater than the sight distance (s).

As previously determined by equation 7, the initial offset at 1 station from point of tangency equals: (see figure 9)

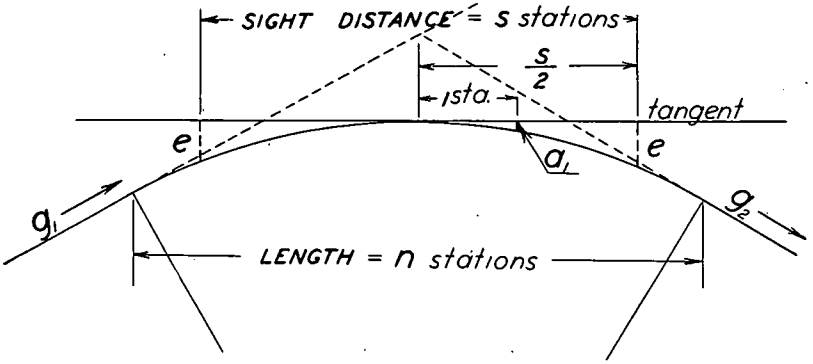
$$a_1 = \frac{(g_1 - g_2)}{2n} \tag{7}$$

The offset for the operator's eye (e), then may be computed as:

$$e = \frac{(g_1 - g_2)}{2n} \times \left(\frac{S}{2}\right)^2 \text{ or } e = \frac{S^2(g_1 - g_2)}{8n} \tag{23}$$

Which may be rewritten as:

$$n = \frac{S^2(g_1 - g_2)}{8e} \tag{24}$$



SUMMIT SIGHT DISTANCE WHEN
LENGTH OF VERTICAL CURVE IS GREATER THAN SIGHT DISTANCE

FIGURE 9

As previously stated, the value of $e = 5.0'$ has been substituted in equation 24, and then written in logarithmic form:

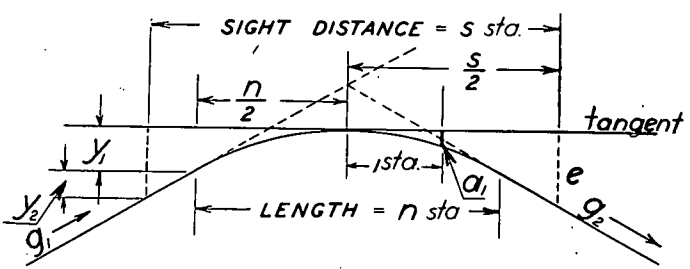
(25) $$\text{Log. } n = 2 \text{ Log. } S + \text{Log. } (g_1 - g_2) - \text{Log. } 40$$

Case II. Figure 10 represents case II, when the length of the vertical curve is shorter than the sight distance, where the height of the operator's eye (e) may be expressed as:

(26) $$e = y_1 + y_2$$

From equation 7, a equals:

(7) $$a_1 = \frac{(g_1 - g_2)}{2n}$$



SUMMIT SIGHT DISTANCE WHEN
LENGTH OF VERTICAL CURVE IS SMALLER THAN SIGHT DISTANCE

FIGURE 10

Then the offset y_1 may be calculated as:

$$(27) \qquad y_1 = \frac{(g_1 - g_2)}{2n} \left(\frac{n}{2}\right)^2 \quad \text{or} \quad \frac{n^2(g_1 - g_2)}{8n}$$

The value y_2, vertical leg of a right triangle, equals:

$$(28) \qquad y_2 = \frac{(g_1 - g_2)}{2} \left(\frac{s - n}{2}\right)$$

When equations 27 and 28 are substituted in equation 26 we have:

$$(29) \qquad e = \frac{n(g_1 - g_2)}{8} + \frac{(g_1 - g_2)\,(s - n)}{4}$$

Which may be solved for in terms of n as:

$$(30) \qquad n = s - \frac{8e}{(g_1 - g_2)}$$

Equation 30 as shown is arithmetically linear, therefore would plot as a straight line on ordinary arithmetic graph paper.

Equations 25 and 30 are both plotted on the Log. Log plot, figure 11, having the logarithm of the sight distance in feet as an ordinate and the logarithm of the algebraic difference in grades on the abscissa. The equation 25 plots as a straight line and the results are shown to the right and above the straight diagonal line, while equation 30 plots as a curved line and its results are shown to the left and below the straight diagonal line. Equations 25 and 30 are equal to each other when n equals S, and these equations are equal to each other along the same straight diagonal line.

The sag sight distance has been developed upon the premise, that the rays of the head light are horizontal or parallel to the roadway (at any rate the rays from head light are not inclined upward). The distance d is the height of the head lamp above the roadway, as shown in figure 12. The initial offset at one station from the point of tangency is expressed by equation 7 as:

$$(7) \qquad a_1 = \frac{(g_1 - g_2)}{2n}$$

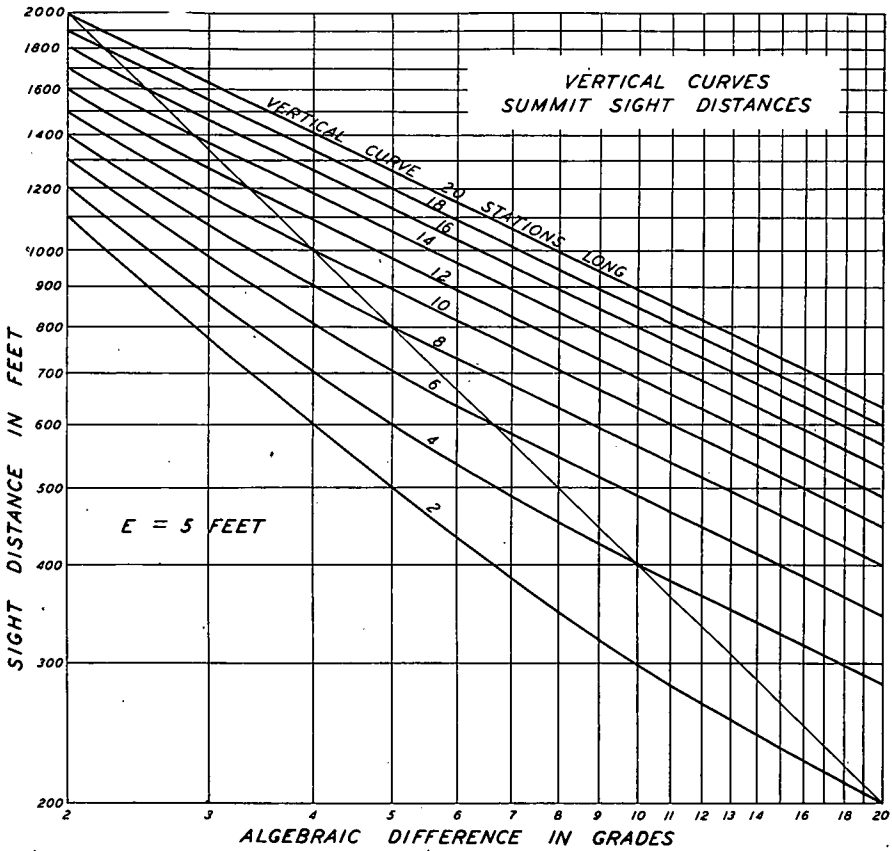
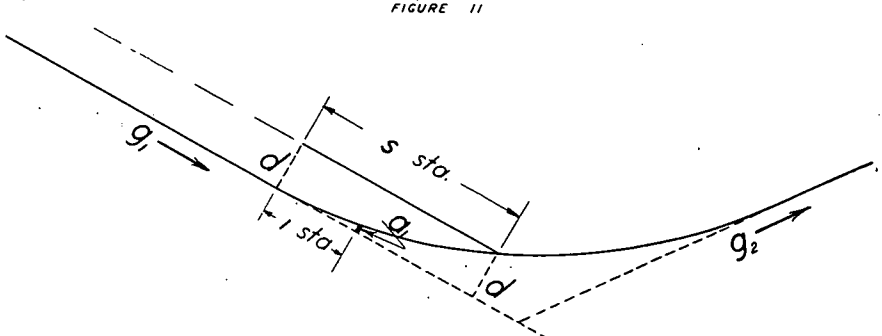


FIGURE 11



SAG SIGHT DISTANCE
NIGHT DRIVING

FIGURE 12

The offset d, expressed in terms of S, and a, becomes:

$$d = \frac{S^2(g_1 - g_2)}{2n} \tag{31}$$

From which:

$$n = \frac{S^2(g_1 - g_2)}{2d} \tag{32}$$

Naturally, the value of d varies with different vehicles, but in this paper for discussion purposes d has been assumed to be 2.5 ft. and then equation 32 becomes logarithmically:

$$\text{Log. } n = 2 \text{ Log. } S + \text{Log. } (g_1 - g_2) - \text{Log. } S. \tag{33}$$

The results of equation 33 have been plotted in figure 13, with

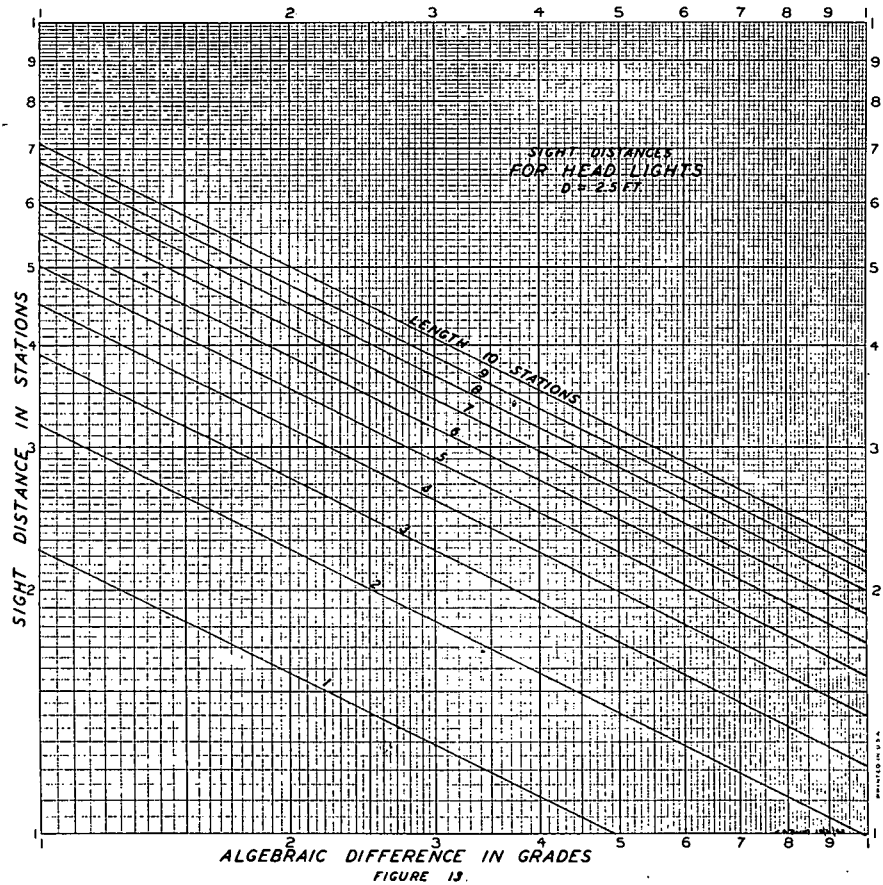


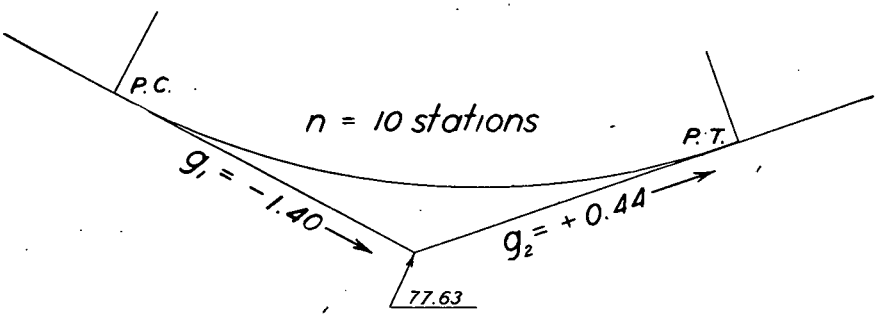
FIGURE 13.

the algebraic difference in grades plotted along the abscissa and the sight distance in stations plotted as an ordinate.

Numerical Examples

Example 1. The following solutions are based upon these given data: $g_1 = -1.40$; $g_2 = +0.44$; $n = 10$ stations; and vertex elevation $= 77.63$. By differential leveling the elevation of the P. C. and P. T. becomes:

$$\text{Elev. P. C.} = 77.63 + 1.40 \times 5 = 84.63$$
$$\text{Elev. P. T.} = 77.63 + 0.44 \times 5 = 79.83$$



Problem No. 1.

The initial offset a may be computed by equation 7:

$$a_1 = \frac{(g_1 - g_2)}{2n} = \frac{-1.40 - (+0.44)}{2 \times 10} = -0.0920$$

This offset a_1 may, also, be found from the graph shown as figure 4. The ordinate values have a range for an algebraic difference in grades of 2.0 to 20.0. This illustrated problem has an algebraic difference in grades equal to 1.84. It, therefore, becomes necessary to move the decimal point one place to the left for all ordinate values, and at the same time move the decimal point one place to the left for all abscissa values (a_1). When these decimal point changes are observed, a value of $a_1 = 0.092$ is readily obtained from figure 4.

The offset a may be obtained from Table I, as shown below:

	$(g_1 - g_2)$	a_1
	0.04	0.00200
	0.80	0.04000
	1.00	0.05000
when $(g_1 - g_2) = 1.84$		0.09200 $= a_1$

Solution by the A.R.E.A. method:

Elev. sta. 0 =	84.630	g_1 =	—1.400
$(g_1—a_1)$ =	—1.308	a_1 =	—0.092
Elev. sta. 1 =	83.322	$(g_1—a_1)$ =	—1.308
$(g_1—3a_1)$ =	—1.124	$2a_1$ =	—0.184
Elev. sta. 2 =	82.198	$(g_1—3a_1)$ =	—1.124
$(g_1—5a_1)$ =	—0.940	$2a_1$ =	—0.184
Elev. sta. 3 =	81.258	$(g_1—5a_1)$ =	—0.940
$(g_1—7a_1)$ =	—0.756	$2a_1$ =	—0.184
Elev. sta. 4 =	80.502	$(g_1—7a_1)$ =	—0.756
$(g_1—9a_1)$ =	—0.572	$2a_1$ =	—0.184
Elev. sta. 5 =	79.930	$(g_1—9a_1)$ =	—0.572
$(g_1—11a_1)$ =	—0.388	$2a_1$ =	—0.184
Elev. sta. 6 =	79.542	$(g_1—11a_1)$ =	—0.388
$(g_1—13a_1)$ =	—0.204	$2a_1$ =	—0.184
Elev. sta. 7 =	79.338	$(g_1—13a_1)$ =	—0.204
$(g_1—15a_1)$ =	—0.020	$2a_1$ =	—0.184
Elev. sta. 8 =	79.318	$(g_1—15a_1)$ =	—0.020
$(g_1—17a_1)$ =	0.164	$2a_1$ =	—0.184
Elev. sta. 9 =	79.482	$(g_1—17a_1)$ =	+0.164
$(g_1—19a_1)$ =	0.348	$2a_1$ =	—0.184
Elev. sta. 10 =	79.830	$(g_1—19a_1)$ =	+0.348

*(Editor's Note: Professor Baird has prepared a table giving the values of vertical offsets for each station up to 20 stations based on values of a, from 0.0100 to 0.1000 by 0.0001. This table covers 26 pages and because of the expense of reproduction could not be published with this paper. Photostatic copies of this table may be had at cost from the Society.)

If equation 12 be used, the location of the point of minimum elevation may be found as:

$$x = \frac{g_1 \cdot n}{(g_1 - g_2)} = \frac{-1.40 \times 10}{-1.84} = 7+60.87 \text{ stations}$$

When the value of 7+60.87 stations is substituted for x in equation 4, the offset for the point of minimum elevation becomes:

$$a_x = a_1 x^2 = -0.092(7.6087)^2 = -5.3621$$

And by application of equation 9, the elevation of the lowest point on the curve is:

$$\text{Elev. } x = \text{Elev. P. C.} + g_1 \cdot x - a_x$$
$$\text{Elev. } x = 84.63 - 1.40(7.6087) + 5.3261 = 79.30$$

An examination of the above numerical solution will show that $g_1 \cdot x$ and $-a_x$ are in the ratio of 2.0 to 1.0, for $g_1 \cdot x = -10.6522$ and $-a_x = +5.3261$.

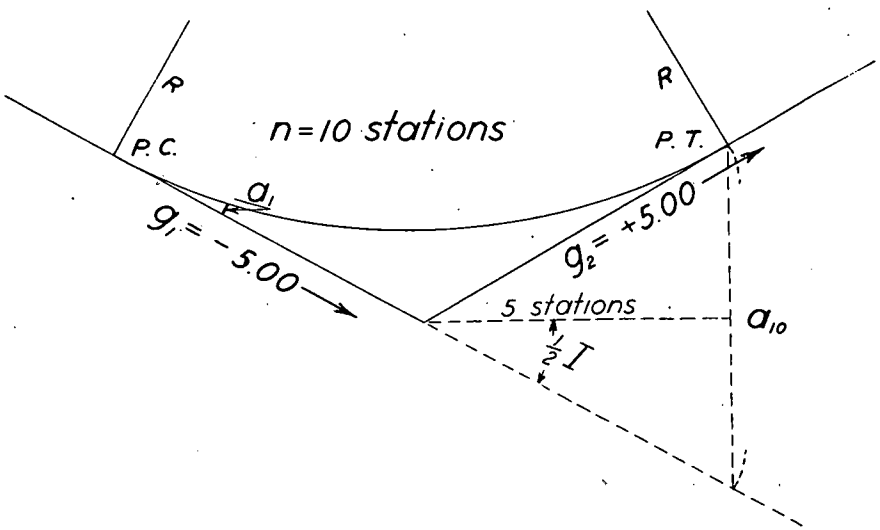
The magnitude of the offset values may be considerably reduced by computing the elevation of each station along the first tangent until the vertex is reached, and then computing the elevation of each station along the second tangent from the vertex to the P. T.

Example 2. This problem compares offsets computed for a parabolic with offsets computed for a circular arc, and the radius of a circular arc with the approximate equation for radius of parabolic vertical curve. The data used in making this comparison are:

$g_1 = -5.00$; $g_2 = +5.00$; and $n = 10$ stations.

The offset a_{10} as calculated by differential leveling:

$$a_{10} = 2 \times 5 \times 5 = 50.000 \text{ ft.}$$



PROBLEM NO. 2.

The offset a_{10} as calculated by vertical curve methods using equations 7 and 6:

$$a_1 = \frac{(g_1 - g_2)}{2n} = \frac{-5.00 - 5.00}{20} = -0.5000 \text{ ft.}$$

$$a_{10} = a_1 \cdot n^2 = -0.5000 \times 10^2 = -50.000 \text{ ft.}$$

The central angle of a circular arc is:

$$\text{Tan}\tfrac{1}{2}I = \frac{5 \times 5}{500} = 0.05000000 \qquad (2^\circ\text{-}51'\text{-}45'')$$

Where:

$$T = R \cdot \tan\tfrac{1}{2}I$$

When:

$$\frac{500}{\text{Cos}(2^\circ\text{-}51'\text{-}45'')} = R \times 0.05000000$$

From which:

$$R = \frac{500}{0.99875226 \times 0.05000000} = 10{,}012.492 \text{ ft.}$$

The offsets a_1 and a_{10} may, also, be computed as offsets y_1 and y_2 as developed in equation 2:

$$y_1 = R - \sqrt{R^2 - X^2_1} = 10{,}012.492 - \sqrt{(10{,}012.492)^2 - (100)^2} = 0.500 \text{ ft.}$$

$$y_2 = R - \sqrt{R^2 - X^2_2} = 10{,}012.492 - \sqrt{(10{,}012.492)^2 - (1000)^2} = 50.063 \text{ ft.}$$

When the approximate equation 17 is used the value of R solves as:

$$R = \frac{(100)^2 \cdot n}{(g_1 - g_2)} = \frac{(100)^2 \times 10}{-10} = -10{,}000 \text{ ft.}$$

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FREQUENCY OF MINOR FLOODS

By Harold A. Thomas, Jr., Member*

In the design of many hydraulic structures it is economical to take a calculated risk of occasional inundation by minor floods rather than to provide complete flood-protection. Such structures include cofferdams, diversion channels, drainage ditches, storm sewers, inverted siphons, levees adjoining unimportant properties, waterpower and water-supply intakes, and sewage outfalls. Also in this class are appurtenances to large, important dams, such as trash racks, baffle-piers, and other energy-dissipating devices downstream that may be damaged occasionally to an extent not impairing the safety of the dam itself. The key factor in balancing the cost of protection against the risk of damages to these structures is the expected frequency of occurrence of the inundating flood. Here the probability method of flood estimation affords a rational basis of design—indeed, it is the only method of flood estimation that relates flood magnitude to recurrence interval.

In the design and operation of another class of structures, including waterpower and water-supply reservoirs, a knowledge of the frequency of periods of low stream flow is of first importance in the intelligent use of water resources. Here again the probability method provides an essential approach to economical design and operation.

The probability method of flow estimation, however, has fallen into disrepute as a tool of the civil engineer. This stems from the fact that the method has not yielded satisfactory results in one application, that of predicting the frequency of major floods. Other, more reliable methods of estimating large floods have superseded the probability method in the design of important structures the failure of which would be disastrous. These methods, however, provide no indication of the time intervals to be expected between floods and accordingly are of little aid to the designer of structures that are expected to be subjected to occasional inundation. Information as to the magnitude of the huge runoff resulting from the theoretical maximum

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possible precipitation over a watershed is indeed of no value in fixing the size of a sinking fund or an insurance rate against the contingency of flood damage.

It is therefore pertinent to investigate the limitations of the probability method that have incurred the current disapprobation and to assess the merits of the method as applied to an important group of engineering problems involving the frequency of minor floods.

Limitations of the Probability Method

A probability-plot of typical flood runoff data for a small stream[1] is shown in Figure 1. Each plotted point represents the maximum annual 24-hour runoff for a particular year during a 25-year period.

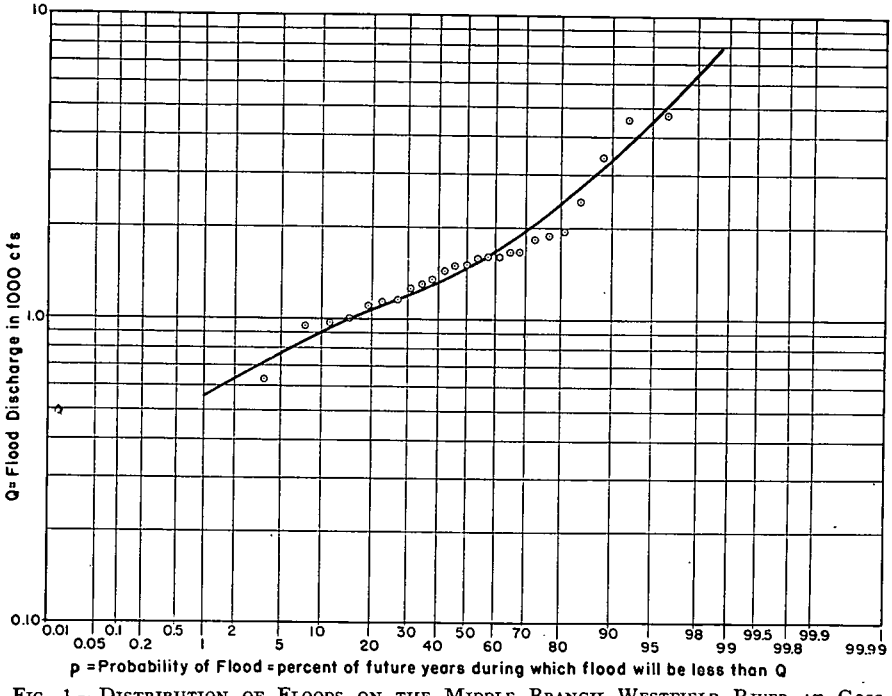

Fig. 1.—Distribution of Floods on the Middle Branch Westfield River at Goss Heights, Mass.

Each point represents the maximum 24-hour runoff for one year during 25 year period 1921-45.

[1]Middle Branch Westfield River at Goss Heights, Mass. The chronological record of maximum annual 24-hour runoff (cfs), 1921-1945, is as follows: 1600, 1930, 1360, 1160, 1600, 970, 1000, 2450, 1490, 629, 1100, 1130, 3390, 1570, 1510, 4550, 1840, 4720, 1260, 1650, 950, 1300, 1660, 1870, 1440.

The maximum annual 24-hour runoff is defined as a flood irrespective of its magnitude.

The selection of the plotting positions of the floods on the probability-scale is somewhat arbitrary and a number of different methods of determining the positions are used. A simple plotting method will be described subsequently that is believed to have a greater degree of theoretical validity than other methods in use. However, no method is entirely correct; the precise establishment of the correct plotting positions is possible only with reliable stream records of extremely long duration, and these do not exist at present.

The essence of the probability method lies in the construction of a line passing through the plotted points to represent the theoretical frequency distribution of floods of different magnitudes. The plotted points in Figure 1 exhibit irregularities of position in their sequence that are generally characteristic of probability-plots of flood records. These irregularities are to be attributed in part to non-significant random fluctuations that will differ in each successive 25 years of record. They are also to be attributed to inherent and significant runoff characteristics of the stream. However, there is no way of determining from records of the usual length whether or not the irregularities are significant.

The construction of the theoretical frequency distribution may be accomplished merely by drawing a curve by eye through the points, or it may be done analytically by rather laborious procedures. In either case, construction is aided by the use of various types of probability-scales—these tend to linearize the positions of the plotted points, facilitating interpolation and extrapolation.

The concept underlying the probability method is that a theoretical frequency distribution exists for each stream and that it will retain substantially the same form over a time comparable to the useful life of hydraulic works. Any change in climatological factors, any physical alteration in the watershed or stream channel may alter the form of the theoretical frequency distribution. In some streams the effect of shifting conditions is such that the probability concept retains little validity except for short periods of time; in other streams there is a considerable degree of stability, and the probability method properly applied will yield useful estimates of flood frequency over long periods of time.

The central difficulty heretofore in the application of the proba-

bility method has been that no satisfactory procedure has been developed for determining the theoretical frequency distribution to which the flood record on a stream would correspond. The error entailed in fitting a theoretical curve to stream flood data becomes especially large in the case of major floods. Indeed, in so far as extrapolation made beyond the range of the recorded floods is concerned, the most elaborate analytical procedure of curve-fitting gives results that are no more reliable than those obtained by a simple extension by eye of the flood frequency curve on any kind of probability paper.

Many types of theoretical frequency distribution have been proposed, and marked discrepancies have been noted among these as to the predicted frequencies of major floods. With smaller floods the agreement is better but not complete. Such discrepancies, coupled with the lack of any rational basis for selecting one theoretical frequency distribution in preference to the others, have been an important factor in the discrediting of the probability method.

However, notable advances have recently been made in the theory of mathematical statistics that eliminate a number of the difficulties that have been encountered in the application of probability theory to hydrological data. The utility of these new statistical techniques has been strikingly demonstrated in other areas of engineering interest —for example, in the quality control of manufactured products.

Foremost among the theoretical innovations has been the development of methods of making useful inferences from observed data without the necessity of assuming that the data are represented by any particular theoretical frequency distribution. These methods, which are called non-parametric, lead to inferences that are valid irrespective of the form of the theoretical distribution from which the observations arise; they apply with equal precision to all possible theoretical distributions. Since non-parametric methods are free from the chief limitation of older (parametric) methods of statistical analysis that were predicated upon the arbitrary assumption of a particular theoretical frequency distribution, the development of a suitable non-parametric method of flood prediction would appear to have immediate utility.

Flood Probability

Some insight into the matter may be had by disregarding for the moment the inherent difficulty of determining exact probabilities of floods of various magnitudes from stream flow records. Assume, for example, that the theoretical probabilities for floods of all sizes for a particular stream are precisely known. Let it be supposed that the smooth curve on Figure 1 is indeed a true representation of the theoretical frequency distribution of floods on the Middle Branch Westfield River. To fix the ideas further, assume that a hydraulic structure designed to withstand flows up to 2400 cfs is to be built. From Figure 1 it may be noted that this design flood has a probability p of 0.80. This implies that during any year there exists an 80% chance that the structure will not be inundated and a 20% chance that it will. Over a long period the structure will be inundated during 20% of the years.

In planning for the maintenance of the structure after construction, the following probability theorem is useful.

The probability that during the next t years the inundating flood will be exceeded exactly k times is given by the following expression:

$$P_k = \binom{t}{k} p^{t-k} (1-p)^k \qquad (1)$$

where $\binom{t}{k} = \frac{t!}{k!\,(t-k)!}$, the binomial coefficient.[2]

The significance of the theorem may be made evident by examples.

Example (i). The probability p is 0.80 that a flood of 2400 cfs will not be exceeded during any year. What is the probability that during the next ten years this flood will not be exceeded?

Here $t = 10$, $k = 0$. Substituting in Equation (1),

$$P_0 = \binom{10}{0} (0.80)^{10} (0.20)^0 = 0.107.$$

Thus the required probability is only 10.7%. An equivalent alternate interpretation is that in 10.7% of future decades the maximum flood per decade will be less than 2400 cfs.

Example (ii). With the data of the preceding example, find the probability that during the next ten years the number of floods exceeding 2400 cfs will not be greater than two.

There are only three ways that the number of floods exceeding 2400 cfs

[2]The binomial coefficient occurs frequently in the ensuing theory and a short set of numerical values is presented in Table 1 for ease of reference. A more complete table appears in "Probability and Its Engineering Uses," Thornton C. Fry, D. Van Nostrand, 1928.

TABLE 1.—BINOMIAL COEFFICIENTS

$$\binom{t}{k} = \frac{t!}{k!\,(t-k)!}$$

k	t = 1	2	3	4	5	6	7	8	9	10
0	1	1	1	1	1	1	1	1	1	1
1	1	2	3	4	5	6	7	8	9	10
2		1	3	6	10	15	21	28	36	45
3			1	4	10	20	35	56	84	120
4				1	5	15	35	70	126	210
5					1	6	21	56	126	252
6						1	7	28	84	210
7							1	8	36	120
8								1	9	45
9									1	10
10										1

k	11	12	13	14	15	16	17	18	19	20
0	1	1	1	1	1	1	1	1	1	1
1	11	12	13	14	15	16	17	18	19	20
2	55	66	78	91	105	120	136	153	171	190
3	165	220	286	364	455	560	680	816	969	1140
4	330	495	715	1001	1365	1820	2380	3060	3876	4845
5	462	792	1287	2002	3003	4368	6188	8568	11628	15504
6	462	924	1716	3003	5005	8008	12376	18564	27132	38760
7			1716	3432	6435	11440	19448	31824	50388	77520
8					6435	12870	24310	43758	75582	125970
9							24310	48620	92378	167960
10									92378	184756

can be not greater than two—namely, when the number is (a) zero, (b) exactly one, (c) exactly two. The required probability of these mutually exclusive events is the sum of the probabilities of each.

$$P_0 + P_1 + P_2 = \binom{10}{0}(0.80)^{10}(0.20)^0 + \binom{10}{1}(0.80)^9(0.20)^1 + \binom{10}{2}(0.80)^8(0.20)^2 = 0.1074 + 0.2684 + 0.3020 = 0.678.$$

About a 1/3 chance exists that there will be more than two floods exceeding 2400 cfs during the next decade. In about 2/3 of future decades (67.8%) the number of floods greater than 2400 cfs will be two or less per decade.

Example (iii). Using the result of Example (ii), find the probability that during the next century *every* decade will have no more than two floods exceeding 2400 cfs.

Shifting the time scale of Equation (1) from a one-year to a ten-year base, $t = 100/10 = 10$, $k = 0$, $p = 0.678$.

$$P = \binom{10}{0} (0.678)^{10}(1-0.678)^{0} = 0.0205.$$

Only a rather small probability (2.05%) exists that during *all* of the next ten decades the number of floods per decade will not exceed two. It is quite likely that there will be a number of these decades during which a flood of 2400 cfs will be exceeded three or more times.

Results of additional calculations based on Equation (1) are presented in Figure 2a to implement an understanding of the relation between the probability of a flood and the frequency with which it occurs.

Return Period of Floods

Ideally the record of annual floods upon a stream may be conceived as an indefinitely long sequence in which each item is characterized as being either greater or less than a certain fixed value, namely, the design flood. Let the symbol E denote the yearly event "no flood," that is to say, a flood less than the design flood, and let

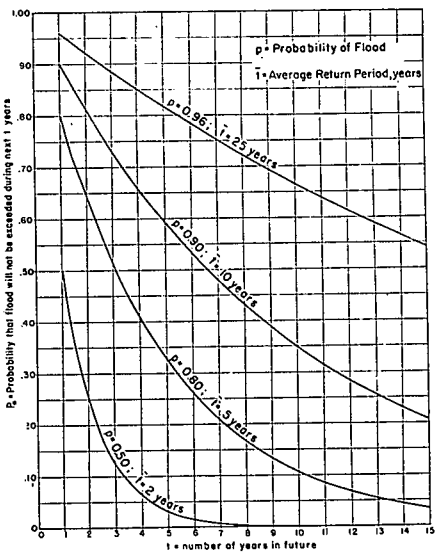

Fig. 2A.—Probability That Different-Size Floods Will Not Occur During Next t Years.
Size of flood stated in terms of average return period.

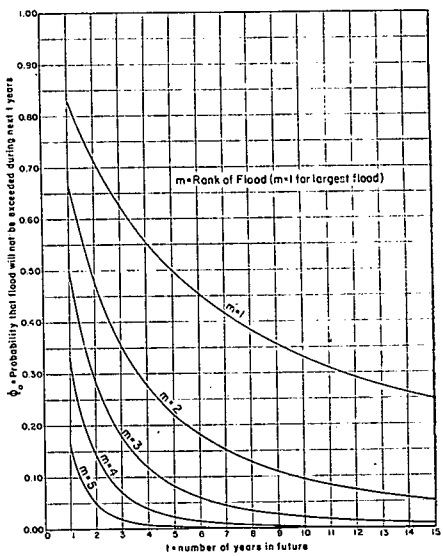

Fig. 2B.—Probability That Different-Size Floods Will Not Recur During Next t Years.
Size of flood stated in terms of recorded discharge of floods that occurred during past 5 years.

the symbol E denote the event "flood," meaning a flow greater than design flood. Then a typical stream flood record may be expressed symbolically as a random sequence—for instance,

$$E\,\mathrm{E}\,\mathrm{E}\,E\,\mathrm{E}\,\mathrm{E}\,\mathrm{E}\,\mathrm{E}\,E\,E\,\mathrm{E}\,\mathrm{E}\,\mathrm{E}\,\mathrm{E}\,E\,E\,\mathrm{E}\,\mathrm{E}\,\mathrm{E}\,\mathrm{E}\,\mathrm{E}\,\mathrm{E}\,\mathrm{E}\,E \cdots .$$

The length of interval between floods, E, is called the return period and varies in an irregular, random manner. In the foregoing sequence the return period takes successive values 3, 5, 2, 4, 1, 8, $\cdots$. Some intervals are long, some short; the minimum return period is one year, the maximum return period theoretically has no upper limit. The larger the design flood, the rarer becomes the event E and the greater the length of the return periods generally. For a fixed value of design flood the intervals represent a random sample of numbers that may be analyzed statistically.

An idea of the theoretical frequency distribution of return periods of different duration may be had by means of the following considerations that pertain specifically to a design flood of 2400 cfs with a probability p of 0.8000. If the probability is 0.8000 that during any one year the design flood will not be exceeded, then

(1) the probability that it will not be exceeded during a particular year and will then be exceeded during the next year is $0.8000(1-0.8000) = 0.1600$;

(2) the probability that the design flow will not be exceeded for two consecutive years and will then be exceeded during the third year is $(0.8000)(0.8000)(1-0.8000) = 0.1280$;

(3) the probability that the design flow will not be exceeded for $t-1$ years and will then be exceeded during the tth year is $(0.8000)^{t-1}(1-0.8000)$.

In Table 2 is presented a compilation of the relative frequencies of return periods of various lengths between floods exceeding 2400 cfs.

The interpretation of the probabilities in Table 2 is as follows: 20% of intervals between floods exceeding the design flood have a duration of one year, 16% a duration of two years, 12.80% a duration of three years, etc. About 1.15% of the intervals exceed 20 years.

The *average* length of return periods between floods exceeding the design value of 2400 cfs may be calculated from the following infinite series:

$$\bar{t} = 1(0.2000) + 2(0.1600) + 3(0.1280) + \cdots ,$$
$$\bar{t} = 5 \text{ years.}$$

In a more general way, the average return period $\bar{t}$ corresponding to a design flood of probability p may be determined from the following series:

$$\bar{t} = 1(1-p) + 2p(1-p) + 3p^2(1-p) + \cdots + tp^{t-1}(1-p) + \cdots ,$$
$$\bar{t} = 1/(1-p) \qquad (2)$$

TABLE 2.—THEORETICAL DISTRIBUTION OF RETURN PERIODS BETWEEN FLOODS EXCEEDING A FLOOD WITH A PROBABILITY OF 0.8000

Return Period, t years	Probability*	Return Period, t years	Probability
1	0.2000	11	0.0215
2	0.1600	12	0.0172
3	0.1280	13	0.0137
4	0.1024	14	0.0110
5	0.0819	15	0.0088
6	0.0656	16	0.0070
7	0.0524	17	0.0056
8	0.0420	18	0.0045
9	0.0335	19	0.0036
10	0.0269	20	0.0029
		>20	0.0115
		Total =	1.0000

*Probability = $(p)^{t-1}(1-p) = (0.8)^{t-1}(0.2)$

The relative frequencies of floods of different magnitudes may be expressed either in terms of their probability or in terms of their *average* return period. Thus a 100-year flood has a probability of 0.99, a 1000-year flood a probability of 0.999, etc.

It is pertinent to note from Table 2, however, that the mere knowledge of the *average* return of a design flood does not give an adequate indication of the magnitudes of the *actual* return periods that may occur. In fact, it is seen that the relative frequency of return intervals of *exactly* 5 years between floods exceeding 2400 cfs is 0.0819. Although the average interval is 5 years, only a little over 8% of actual intervals have a duration of exactly 5 years. Particularly significant is the fact that intervals shorter than the average may be expected to occur more frequently than those greater than the average. Indeed, it may be shown that 10% of the intervals will be less than a value given approximately by the following expressions:

$$t_{10} = \log(1-.10)/\log p = 0.0458/(\log 1/p) \qquad (3)$$

From Equation (3) it may be estimated for a structure designed just to withstand the 100-year flood of the stream that a 10% chance exists that this flood will be exceeded within a period of 0.0458/log (1/0.99) = 10.5 years.

It is to be emphasized that the foregoing computations are predi-

cated upon a knowledge of the probability of the design flood. Therefore the value of the computations depends upon the precision with which the parameter p corresponding to a design flood may be estimated. Ordinarily the error of estimate is large. Perception as to the precision attainable in estimating p may be had by considering the distribution of size of the largest floods in consecutive portions of equal duration of a stream flood record.

Distribution of the Largest Flood

If a long consecutive flood record be broken up into subgroups of equal size and each subgroup be arrayed in descending order of magnitude, the result may be represented schematically as follows:

$$\begin{array}{l} Q_1,\ Q_2,\ Q_3,\ \cdots,\ Q_n \\ Q_1',\ Q_2',\ Q_3',\ \cdots,\ Q_n' \\ Q_1'',\ Q_2'',\ Q_3'',\ \cdots,\ Q''_n \\ \cdot\quad\cdot\quad\cdot\quad\cdot\quad\cdot \\ Q_1^{(N)},\ Q_2^{(N)},\ Q_3^{(N)},\ \cdots,\ Q_n^{(N)}, \end{array}$$

where n = the number of years in each subgroup,

N = the number of subgroups.

Q_1 denotes the largest flood in the first n years of record, Q_2 the next largest, etc. Q_n' represents the smallest flood occurring during the interval between the $n+1^{th}$ and the $2n^{th}$ years, that is, the second subgroup. Evidently the largest flood recorded must appear in the first column. The theoretical frequency distribution of the probabilities of the floods in the first column—the largest in each subgroup—may readily be derived. In fact, the derivation may be extended so as to apply to other columns as well as the first.

Let m denote the rank of the floods in each subgroup, Q_1 denoting the largest flood, Q_2 the next largest, Q_m the mth largest, and Q_n the smallest in the subgroup. In a subgroup of n floods the probability $d\theta$ that (n-m) floods will be smaller than a certain value Q, (m—1) floods will be larger than Q, and one flood will fall in the range $Q \pm \frac{1}{2}dQ$ is

$$d\theta = \binom{n}{m} m\, p^{n-m}\, (1-p)^{m-1}\, dp, \tag{4}$$

where p is the probability that a flood of size Q will not be exceeded during any year.

The probability θ that the actual p-value of the mth of n floods

will be less than a certain fixed value, say p_0, may be obtained by integrating Equation (4).

$$\theta = \binom{n}{m} m \int_0^{p_0} p^{n-m} (1-p)^{m-1} \, dp. \tag{5}$$

For the case of the largest floods ($m=1$),

$$\theta = n \int_0^{p_0} p^{n-1} \, dp = p_0^{\,n}. \tag{6}$$

The significance of this theory lies in the fact that although it is not possible to determine precisely the p-value of a flood, it is possible by Equations (5) and (6) to ascertain the likelihood that the actual p-value lies within certain fixed limits.

Example (iv). The largest flood during a 25-year period on the Middle Branch Westfield River was 4720 cfs. Calculate the chance that this flood has an average return period between 20 and 100 years.

By Equation (2), the p-values corresponding to these limits for the average return period are 0.95 and 0.99 respectively. The likelihood that the actual p-value of the 4720-cfs flood falls in the interval 0.95 to 0.99 may be calculated from Equation (6) as follows:

$$\theta_{0.99} - \theta_{0.95} = 0.99^{25} - 0.95^{25} = 0.5004.$$

Approximately a 50% chance (50.4%) exists that the actual p-value of the 4720-cfs flood lies between 0.95 and 0.99. Correspondingly, about a 50% chance (49.96%) exists that the actual p-value lies outside these limits—that is, with an average return period less than 20 years or greater than 100 years.

Calculations similar to the foregoing may be made for floods smaller than the largest on record with Equation (5), using mathematical tables of the integral of that equation, the incomplete Beta-function. As an illustration of results of calculations of this type, data are presented in Table 3 that give an indication of the reliability with which the average return period of the five largest floods of any 25-year flood record may be established.

The significant fact to be noted in Table 3 is that the lesser floods in a record have narrower confidence intervals and therefore are better established as to frequency of occurrence. The largest flood of a record generally has a broad confidence interval, and its average return period may be estimated only very approximately.

TABLE 3.—50% CONFIDENCE LIMITS FOR THE AVERAGE RETURN PERIODS OF THE FIVE LARGEST FLOODS OF A 25-YEAR RECORD

Rank of Flood, m	50% Confidence Limits (years) for the Average Return Period $\bar{t}$ Lower Limit	Upper Limit
1	18	87
2	10	26
3	6.6	14
4	5.1	9.8
5	4.2	7.3

PLOTTING POSITIONS OF FLOODS

The foregoing theory may be applied to the problem of determining appropriate plotting positions on the probability-scale of graphs of flood records (Figure 1). According to Equation (4), the probability that the actual p-value of the largest flood (m=1) lies within a range of $p \pm \frac{1}{2}dp$ is as follows:

$$d\theta = n\, p^{n-1}\, dp.$$

Taking moments of p about the origin zero, the *mean* p-value of the largest of n floods is given by the relation

$$\bar{p} = \int_0^1 p\,d\theta = n \int_0^1 p\, p^{n-1}\, dp = \frac{n}{n+1}. \qquad (7)$$

Thus the mean p-value of the largest of 25 floods is $25/26 = 0.9615 = 96.15\%$; the corresponding return period by Equation (2) is 26 years.

While it is impossible to establish accurately the p-value corresponding to the largest flood in any particular record of n years, it is possible by Equation (7) to determine the mean p-value of the largest floods of a collective of many hypothetical flood records each of n years' duration. In the absence of an exact knowledge of the actual p-value for the particular record at hand, it is reasonable to assume in the construction of probability-plots that the largest flood may be represented by the $\bar{p}$ of the collective as calculated by Equation (7).

The theory underlying Equation (7) may be generalized to apply to the determination of plotting positions of floods other than the largest. For the mth of n floods.

$$\bar{p} = \frac{n-m+1}{n+1}. \qquad (8)$$

For a 25-year record the plotting positions $\bar{p}$ of the floods, starting with the largest, are, by Equation (8), $25/26 = 96.15\%$, $24/26 = 92.31\%$, $23/26 = 88.46\%$, etc. The smallest flood is to be plotted at $\bar{p} = 1/26 = 3.85\%$ (Figure 1).

It should be noted that in deriving Equations (7) and (8) no assumption was made as to any functional relation between p and Q —that is, no particular theoretical frequency function was assumed to apply. The reasoning is valid for all theoretical frequency distributions, and Equation (8) may be used with any type of probability paper.

While an estimate of the probability p of an observed flood may be made with Equation (8) and some idea of the reliability of this estimate may be had with the aid of Equations (5) and (6), it is possible, as has been stated, to eliminate entirely the necessity for estimating p by the use of a non-parametric method. This method provides a more precise basis for flood prediction than that obtained with Equation (1) with estimated values of p.

Non-Parametric Method of Flood Estimation

The practical difficulty in using Equation (1) to predict floods lies in inevitable error entailed in estimating the parameter p. Equation (4), however, provides a rational basis for weighting various values of p as it is varied from 0 to 1. The following integral is the equivalent of the right-hand side of Equation (1) when a *weighted average* of all possible values of p is used.

$$\Phi_k = \int_0^1 P_k d\theta = \binom{t}{k}\binom{n}{m} m \int_0^1 p^{t-k}(1-p)^k p^{n-m}(1-p)^{m-1}\, dp.$$

Integrating,

$$\Phi_k = \frac{m\binom{t}{k}\binom{n}{m}}{(m+k)\binom{t+n}{m+k}}, \tag{9}$$

where Φ_k is the probability that in t future years the mth of n past floods will be exceeded *exactly* k times.

An important special case of the foregoing equation occurs when $k = 0$:

$$\Phi_0 = \frac{\binom{n}{m}}{\binom{t+n}{m}}, \tag{10}$$

where Φ_0 is the probability that in t future years the mth of n past floods will not be exceeded.

For the largest flood (m=1) observed in n past floods,

$$\Phi_0 = \frac{\binom{n}{1}}{\binom{t+n}{1}} = \frac{n}{t+n}. \tag{11}$$

From Equation (11) it is evident that a 50% chance exists that the largest flood of the past will have been exceeded after the lapse of ten future years.

In Figure 2b are plotted solutions of Equation (10) for n=5. Figure 2 is arranged so as to facilitate comparison between the parametric and non-parametric flood frequency formulation. In using Figure 2a p (or $\bar{t}$) must be estimated; in using Figure 2b, nothing is estimated—the quantities involved are all exact. It may be noted from these plots that the largest of 5 past floods may be expected to be nearly as large as a flood having an average return period of 10 years.

Further significance of Equations (9), (10), and (11) may be set forth by examples.

Example (v). A cofferdam designed to withstand flows up to and including the 1944 flow of 1870 cfs is planned for the Middle Branch Westfield River. Investigate the degree of protection afforded by a dam of this size during a 5-year program of channel improvement and dam construction.

The 1944 flood is the sixth largest (m=6) during a 25-year record, 1921-1945 (n=25). The probability Φ_0 that this flood will not be exceeded (k=0) during the next 5 years (t=5) may be calculated as follows from Equation (10):

$$\Phi_0 = \frac{\binom{25}{6}}{\binom{30}{6}} = \frac{25!\,6!(30-6)!}{6!(25-6)!\,30!} = \frac{25!\,24!}{19!\,30!} = \frac{20\cdot21\cdot22\cdot23\cdot24}{26\cdot27\cdot28\cdot29\cdot30} = 0.298$$

The probability is rather high that the flood of 1870 cfs will be exceeded at least once during the next five years, being 100(1—0.298) = 70.2%.

Example (vi). With the data of Example (v), calculate the probability that during the next 5 years the 1944 flood will not be exceeded more than twice.

The required probability is the sum of the probabilities of the mutually exclusive events that the flood will be exceeded (a) not at all (k=0), (b) exactly once (k=1), and (c) exactly twice (k=2). From Equations (9) and (10),

$$\Phi_0+\Phi_1+\Phi_2=\frac{6\binom{25}{6}}{6\binom{30}{6}}+\frac{6\binom{25}{6}}{7\binom{30}{7}}+\frac{6\binom{25}{6}}{8\binom{30}{8}}=0.2983+0.3728+0.2269$$

$$=0.898\ .$$

The probability is rather small, 100(1—0.898) = 10.2%, that the 1944 flood will be exceeded more than twice.

Results of additional computations for the Middle Branch Westfield River are shown in Figures 3 and 4. As a matter of interest,

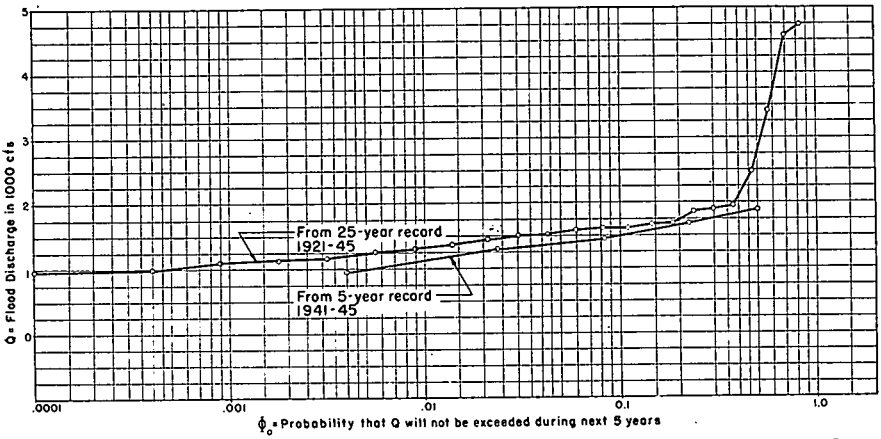

FIG. 3.—PROBABILITY OF NON-RECURRENCE OF FLOODS OF VARIOUS MAGNITUDES Q ON MIDDLE BRANCH WESTFIELD RIVER AT GOSS HEIGHTS, MASS., DURING NEXT 5 YEARS AS BASED UPON 5-YEAR AND 25-YEAR RECORDS.
The two smallest floods during 25-year period are not shown.

results as based upon a flood record of only 5 years (1941-45) are presented in these figures. The 1944 flood was the largest flood (m=1) during 1941-45. The broader range of probability information attainable from the longer record is indicated in Figure 3, in which each point represents a single yearly flood.

VALIDITY OF PROBABILITY CONCEPT

In order to test the reliability with which the frequency of minor floods may be predicted by Equation (9), the records of nine rivers discharging into the Atlantic Ocean were analyzed. The rivers, together with the length of record investigated, are as follows: Penobscot, 30 yrs; Mattawamkeag, 30 yrs; Kennebec, 40 yrs; Merrimack,

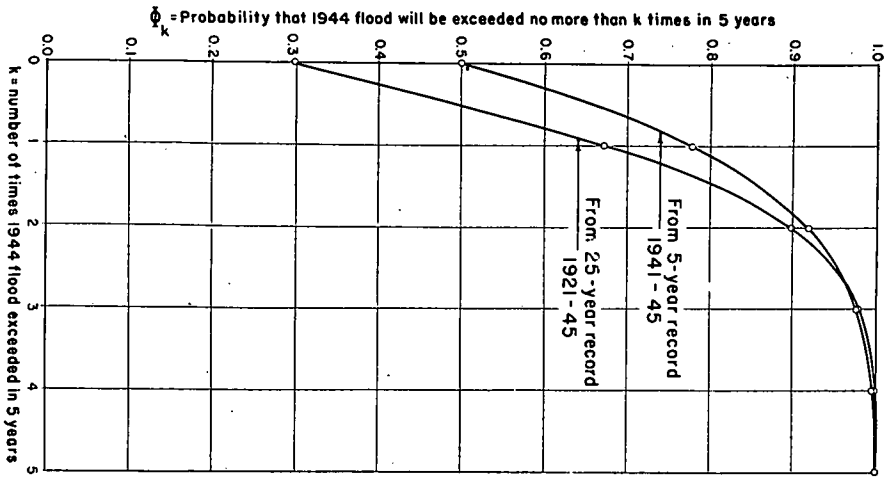

FIG. 4.—PROBABILITY THAT 1944 FLOOD ON THE MIDDLE BRANCH WESTFIELD RIVER AT GOSS HEIGHTS, MASS., WILL BE EXCEEDED NO MORE THAN k TIMES DURING 5 CONSECUTIVE FUTURE YEARS.

30 yrs; Connecticut, 30 yrs; Hudson, 45 yrs; Delaware, 30 yrs; Susquehanna, 40 yrs; Potomac, 35 yrs. All records were divided into subgroups of five years of consecutive floods. The first of these subgroups for each river was taken to represent the "past," and the remaining subgroups the "future." Then, investigating each stream separately, it was recorded how many times the largest flood (m=1) in the "past" (n=5) was exceeded in each of the subgroups (t=5) of the "future." The process was repeated for the second largest flood (m=2) and so on for the lesser floods of the "past."

The results of the investigation for all rivers are compiled in Table 4. To facilitate comparison of actual flood recurrences with those predicted by Equation (9), the latter have been included in the table in parentheses.

$$26.5 = 53 \times \frac{(1)\binom{5}{0}\binom{5}{1}}{(1+10)\binom{10}{1}} = 53\frac{5}{10} = 26.5$$

Interpretation: 26.5/53 = 0.500 is the theoretical probability that the largest of 5 past floods will not be exceeded during the next 5 years. 25/53 = 0.472 is the observed relative frequency.

The agreement between theory and observation for this small random sample of 310 floods from 9 rivers is excellent. Exact agree-

TABLE 4.—COMPARISON OF ACTUAL FLOOD RECURRENCES WITH PREDICTED RECURRENCES BY EQUATION (9).
Predicted values are shown in parentheses. Penobscot, Mattawamkeag, Kennebec, Merrimack, Connecticut, Hudson, Delaware, Susquehanna, and Potomac Rivers.

Number of Times Exceeded, k	Rank of Flood, m				
	1	2	3	4	5
0	25 (26.5)*	14 (11.8)	7 (4.4)	1 (1.3)	0 (0.2)
1	14 (14.7)	19 (14.7)	10 (9.5)	6 (4.2)	1 (1.0)
2	10 (7.4)	12 (12.6)	12 (12.6)	8 (8.4)	2 (3.2)
3	4 (3.2)	6 (8.4)	10 (12.6)	10 (12.6)	9 (7.4)
4	0 (1.0)	1 (4.2)	9 (9.5)	12 (14.7)	12 (14.7)
5	0 (0.2)	1 (1.3)	5 (4.4)	16 (11.8)	29 (26.5)
Total	53 (53)	53 (53)	53 (53)	53 (53)	53 (53)

*By Equation (9), n=5, t=5, m=1, k=0;

ment between observed and theoretical frequencies is of course not to be expected—discrepancies are normal in small samples. Various tests, including the chi-squared test, for statistical significance of the data in Table 4 indicate a degree of agreement that validates the application of the probability concept to the rivers involved.

APPLICATION TO DROUGHTS

The theory embodied in Equation (9) may be used to analyze any type of hydrological phenomena that occur in a fairly random and non-periodic way—as, for instance, the frequency of periods of low stream flow. The scope of the formulation in this respect is indicated by means of an example.

Example (vii). An impounding reservoir is designed so as to have a capacity sufficient to meet the water requirements of a city during a drought year in which the mean rate of stream flow during the filling period is 4.5 cfs. During 30 years of record the lowest flow observed during the filling period was 3.8 cfs (m=30) and the second lowest was 4.5 cfs (m=29). Find the probability that during the next 20 years a flow less than 4.5 cfs (1) will not occur, (2) will occur no more than once, and (3) will occur two or more times.

Here n=30, m=29, t=20.

(1) k=20. By Equation (9),

$$\Phi_{20} = \frac{29\binom{20}{20}\binom{30}{29}}{(29+20)\binom{50}{49}} = 0.355 .$$

(2) k=20,19.

$$\Phi_{20} + \Phi_{19} = \frac{29\binom{20}{20}\binom{30}{29}}{49\binom{50}{49}} + \frac{29\binom{20}{19}\binom{30}{29}}{48\binom{50}{48}}$$

$$= 0.3551 + 0.2959 = 0.651 .$$

(3) k=0,1,2, . . . , 18.

$$\sum_{k=0}^{18} \Phi^k = 1 - \Phi_{19} - \Phi_{20} = 1 - 0.651 = 0.349$$

The probability is not small (35%) that flows less than 4.5 cfs will occur more than once during the next 20 years. A larger reservoir capacity based upon a minimum flow of 3.8 cfs would be more in line with conservative practice, since for this flow a probability of only

$$1 - \frac{30\binom{20}{20}\binom{30}{30}}{50\binom{50}{50}} - \frac{30\binom{20}{19}\binom{30}{30}}{49\binom{50}{49}} = 0.155$$

exists that more than one drought more severe will occur during the next 20 years.

Summary

The probability method of flood prediction, properly applied, will yield useful estimates of the frequency of minor floods. Of the two forms of application of the method, parametric and non-parametric, the latter is to be preferred, since it gives exact, not approximate, flood probabilities.

STRUCTURAL SANDWICH MATERIALS

By A. G. H. Dietz, Member*

(Presented at a meeting of the Structural Section of the Boston Society of Civil Engineers, held on January 14, 1948.)

Sandwiches composed of several layers of different materials, bonded together into a single structural slab, are being employed in increasing quantities for an expanding variety of uses. Such sandwiches are employed because in the composite slabs properties can be obtained which are unobtainable in the constituent materials when used by themselves. Among these properties are high strength-weight ratios, economy, improved insulation, space-saving, and a variety of other properties.

Sandwiches may be classified in a variety of different ways, but for purposes of this paper it may be convenient to classify them as construction sandwiches and transport sandwiches. In the construction field the most important criteria are usually economy and durability, with space-saving and weight reduction secondary, whereas in transport, particularly in military aircraft, weight-saving is by far the most important criterion, so that a high strength-weight ratio is the first consideration. Durability is also important, but the life of an airplane is relatively short compared to most buildings. Cost is almost always secondary to weight saving.

A typical sandwich consists of two relatively thin, strong, dense skins or faces bonded to a relatively thick, lightweight, but considerably weaker core. The skin carries most of the stresses, the core provides the necessary lateral support to keep the skin from wrinkling or buckling under compressive stress, and the geometry of the arrangement provides a high stiffness factor or EI value which makes the arrangement efficient in flexure and in edgewise compression. From the standpoint of building panels, the skin generally provides a wind and weather-resistant face on the outside and a finished face on the inside, while the core provides insulation against heat transfer, sound transmission, or both. For exterior walls such an arrangement can generally provide better heat insulation in considerably thinner sections than do traditional materials like masonry.

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Types of Skins and Cores

Depending upon the end-use, a variety of skin and core materials has been used or tried in experimental installations. Perhaps one of the oldest in the construction field is the combination of cement-asbestos board face on an insulating board core illustrated in Figure 1. This combination has been used in large quantities in both housing and commercial construction for walls, partitions, and roofs. A typical house is shown in Figure 2. Some selected properties of one type of slab employing a bagasse-board (extracted sugar cane) core are given in Table 1.

Other skin materials include plywood, aluminum, steel (plain carbon or alloy), lignocellulose hardboard, and a variety of plastics-based laminates such as paper or fabric impregnated with phenolic or polyester types of resins. The skins may themselves be composites such as plywood or wood veneers faced with resin-impregnated paper or, in table-tops for example, plywood incorporating a sheet of aluminum to provide rapid heat dissipation and to prevent charring from localized heat sources.

A considerable variety of materials may be used for cores in addition to the large variety of insulating boards such as bagasse or wood pulp. These cores include matted glass fiber; foamed glass; foamed hard rubber; foamed plastics like cellulose acetate, polystyrene, phenolic, and others; balsa wood, either side grain or end

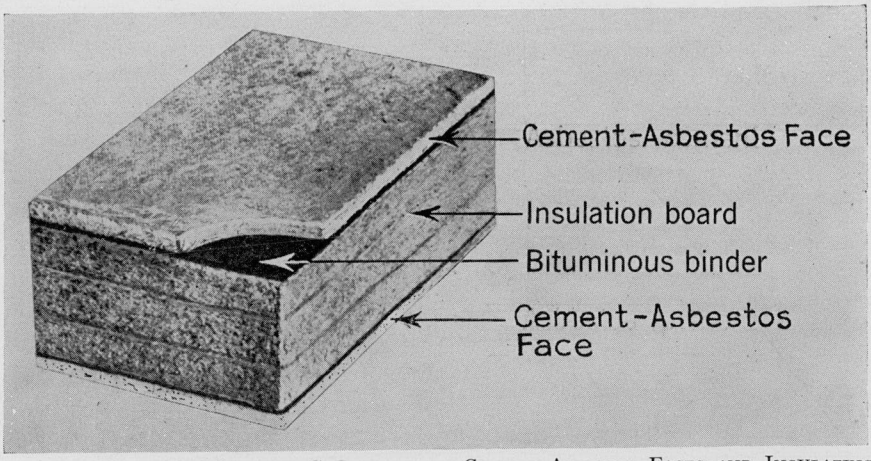


Fig. 1.—Sandwich Material Consisting of Cement-Asbestos Faces and Insulating Board Core.

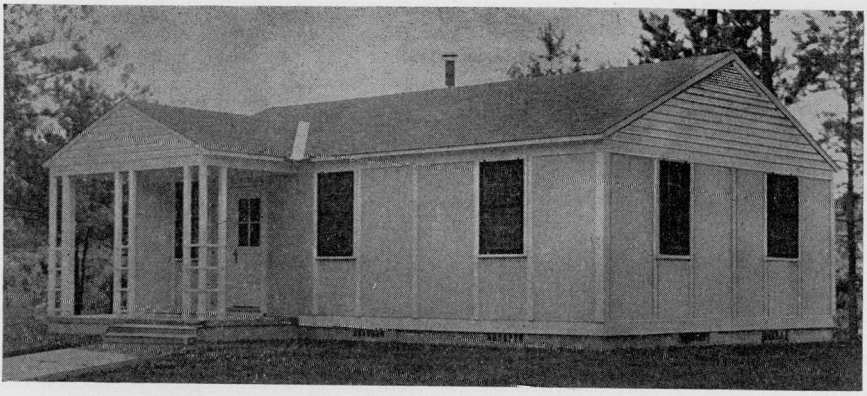


FIG. 2.—HOUSE BUILT OF SANDWICH PANELS.

grain; plywood; and a variety of corrugated or honeycomb-shaped fabricated cores.

TABLE 1.—PROPERTIES OF STRUCTURAL SANDWICH, CEMENT-ASBESTOS FACES, INSULATING BOARD CORE

Overall thickness	1⅛″	1 9/16″	2″
Core thickness	⅞″	1 5/16″	1¾″
Conductance of core (K=0.338Btu/in.)	0.38	0.25	0.19
Thermal conductivity (U) air-to-air			
Inside partitions	0.25	0.19	0.15
Outside walls	0.28	0.20	0.16
Conductance (surface to surface)	0.36	0.24	0.18
Bending			
Load			
4′×8′ panel, supported 4 edges	200 psf		
4′×12′ panel, supported 4 edges		133 psf	
Deflection			
4′×8′ panel, supported 4 edges 75 psf, deflection at center		0.341″	(immediate)
		0.402″	(after 24 hr.)
Tensile, specimens 1 in. wide	212 lb	237 lb	245 lb
Edge compression, per in. width	1235 lb	1700 lb	1790 lb
Lateral nail-holding power			
Nails driven through panel into wooden block. Load per nail			
6d common	259 lb		
8d "	301 lb	241 lb	
10d "		370 lb	
12d "			399 lb

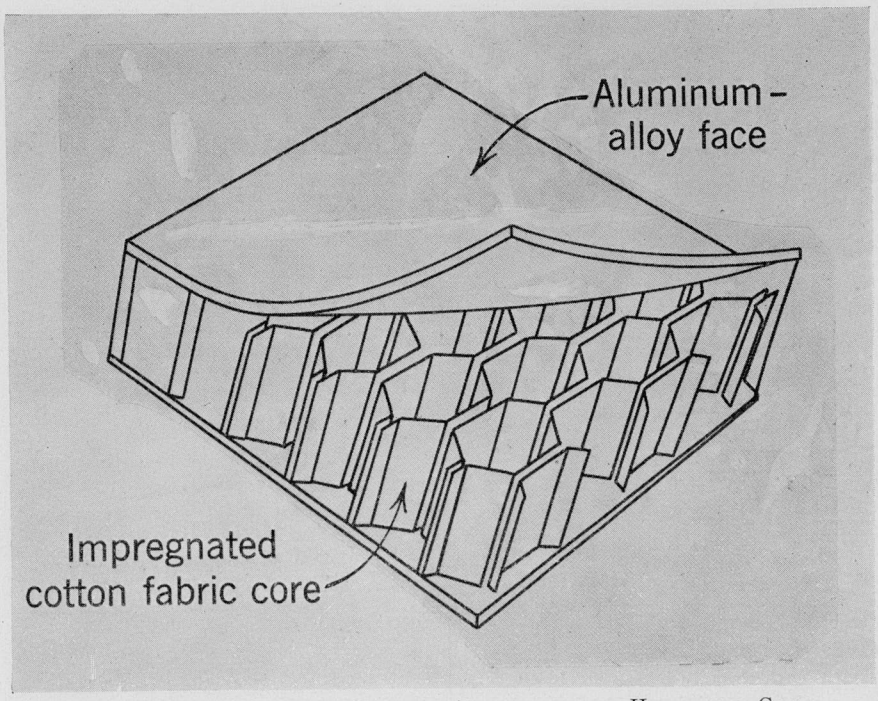


FIG. 3.—SANDWICH PANEL SHOWING ARRANGEMENT OF HONEYCOMB CORE.

In Figures 3, 4, 5, and 6 are shown honeycomb, foamed hard rubber, foamed plastic, and corrugated paper cores. Figure 7 compares the compressive strengths and weights of a number of core materials. Figures 8 and 9 show the assembly of an airplane wing section and the completed wing, employing honeycomb cores.

For the construction field most interest has centered upon foamed glass, foamed hard rubber, matted glass fiber, and the various corrugated and honeycomb-shaped cores in addition to the insulating board types of cores. The mineral-based materials like foamed and matted glass have been combined with metal faces like aluminum and stainless steel for use in commercial buildings, whereas the corrugated, honeycomb, and insulating board cores have been combined with plywood, cement-asbestos board, resin-impregnated paper, and aluminum for use in housing and industrial buildings.

For transportation, particularly in aircraft, the various foamed plastics, foamed rubber, balsa wood, and honeycomb cores have been

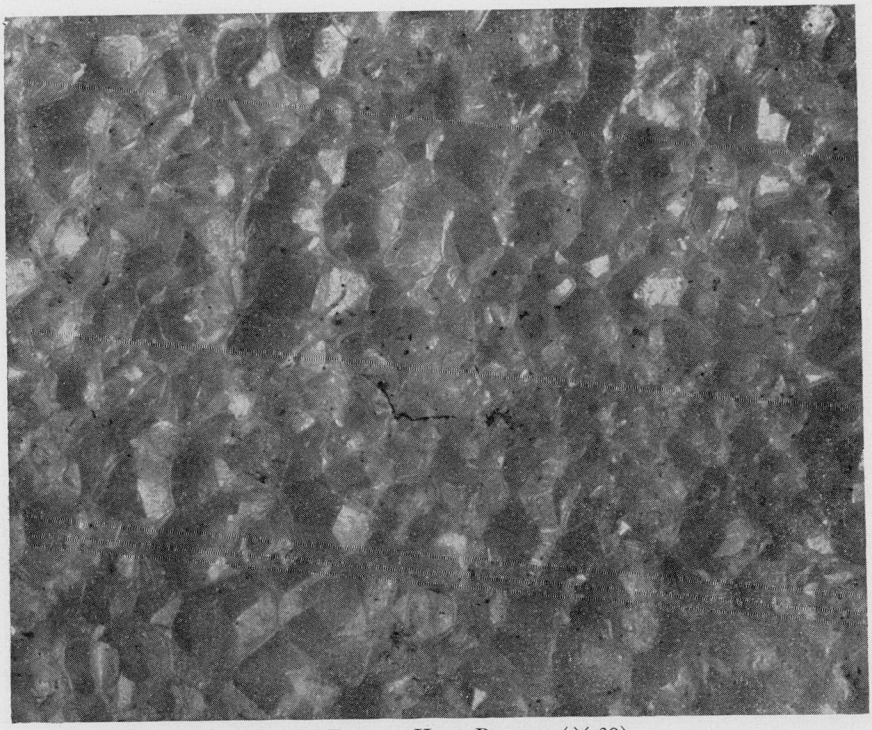


FIG. 4.—FOAMED HARD RUBBER (× 20).

developed in conjunction with skins of aluminum, stainless steel, impregnated paper or fabric, and plywood. A particularly interesting combination is resin-impregnated fiberglas fabric shaped into honeycombs for the core and used also for the skins. These fiberglas-fabric materials develop exceptionally high strength properties and consequently have favorable strength-weight ratios.

ADHESIVES

An important aspect of all sandwich materials is the adhesive employed to bond the core to the skins. Cement-asbestos faces are customarily bonded to insulating-board cores with asphaltic or bituminous binders. Cellular materials like the various foams, corrugated cores, and honeycombs pose a special problem in that the contact between skin and core occurs only along thin edges and ridges of the core and a good bond must be obtained at these limited surfaces.

An attempt is generally made, therefore, to form a fillet of adhesive along the contacting edges in order to increase the effective bonding area, and the formulation of the adhesive must therefore be one

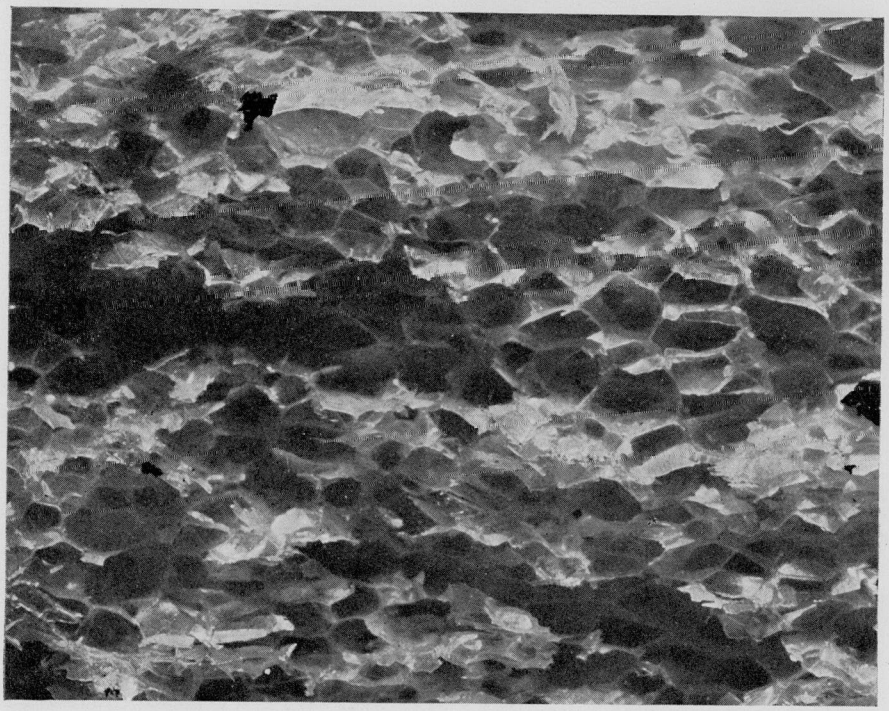
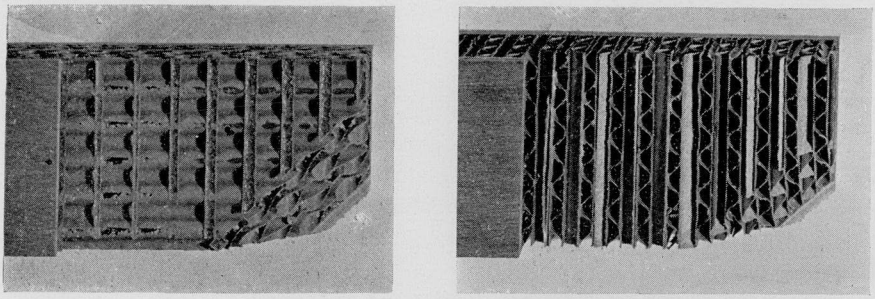


FIG. 5.—FOAMED PLASTIC; CELLULOSE ACETATE (× 20).



A B

FIG. 6.—TWO TYPES OF CORRUGATED PAPER CORES.
(A) All corrugations parallel to skins. (B) Half of cores perpendicular to skins.

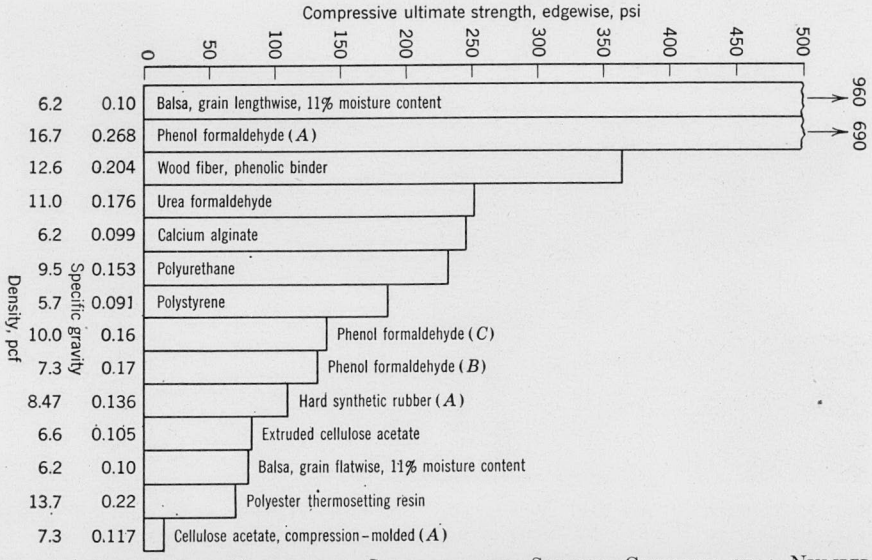


FIG. 7.—COMPARATIVE COMPRESSIVE STRENGTHS AND SPECIFIC GRAVITIES OF A NUMBER OF CORE MATERIALS.

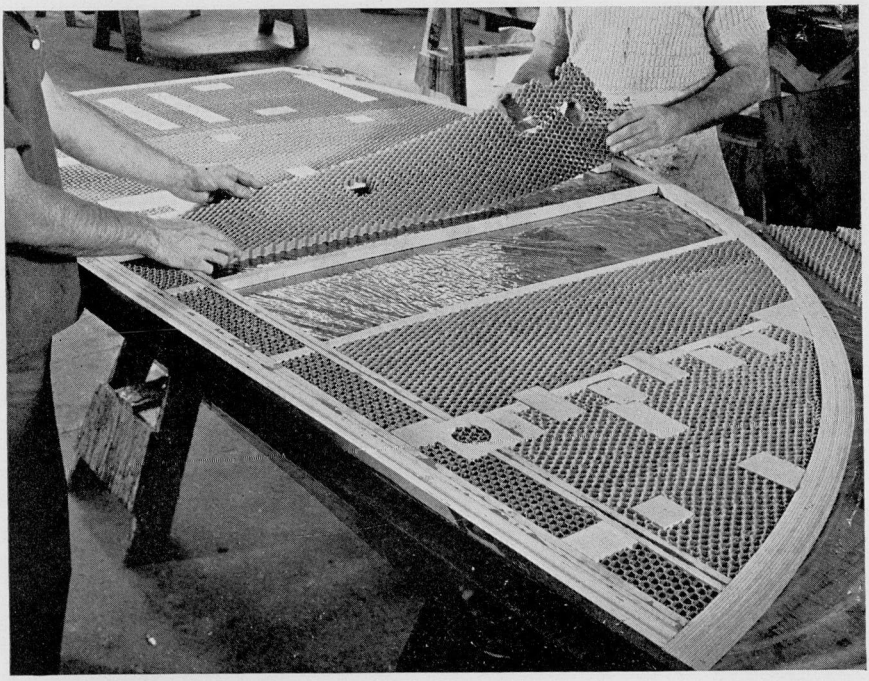


FIG. 8.—HONEYCOMB CORE MATERIAL BEING ASSEMBLED INTO AIRCRAFT WING SECTION.

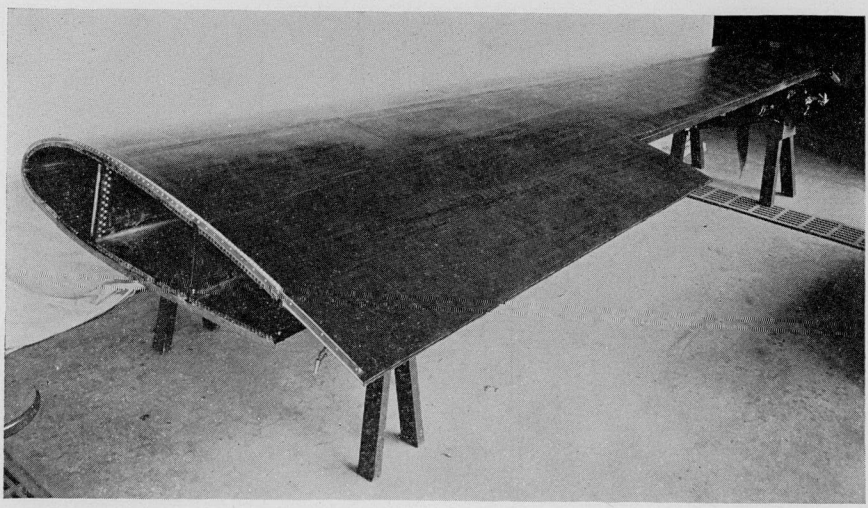


FIG. 9.—COMPLETED AIRCRAFT WING, SHOWING CONSTRUCTION OF SANDWICH TYPE INCORPORATING HONEYCOMB CORES AND SKINS OF PLASTIC-IMPREGNATED FIBERGLAS.

which will tend to form such a fillet. A further important factor is that the coefficients of thermal expansion of skins and cores are likely to be quite different and changes in temperature therefore set up stresses—primarily shear—which the adhesive must be able to absorb without fatigue failure after a considerable number of repetitions of stress occasioned by repeated changes in temperature. Many of the best adhesives must be hardened or "cured" by heat, and heavy stresses may be induced by the large change in temperature occasioned by cooling from the curing temperature to ordinary temperatures. Adhesives must therefore be somewhat flexible. For this reason, some of the favorite formulations employ combinations of thermosetting synthetic resins with elastic constituents like natural and synthetic rubber or with elastomeric plastics.

Bonds between plywood or other cellulose skins and honeycomb, corrugated, or other cellular cellulosic cores can readily be made with low-temperature setting adhesives like urea-formaldehyde, casein, and resorcinol-formaldehyde which can be hardened at ordinary temperatures. Bonds to metals like aluminum and stainless steel usually require high-temperature adhesives whose curing temperatures may range up to 300° F. or higher.

Design Considerations

The structural design of sandwich materials may become fairly complex because the material is not homogeneous and often is non-isotropic, as in plywood faced corrugated cored construction. The usual design formulae for simple beams, slabs, plates, shells, struts, and sheets must therefore be modified to allow for the composite properties of the sandwich. It is far beyond the scope of this brief discussion to explore the design of sandwich materials but the problems involved in simple bending and in edgewise compression may be indicated.

Bending

In most structural design, deflection is calculated with sufficient accuracy by assuming that only the value of Young's modulus E and the moment of inertia I enter into the picture and that the additional deflection caused by shear distortion may be ignored. In sandwich materials the shearing modulus of the core material is generally so low that substantial deflection may occur because of shear alone, and a factor must be introduced to account for it. In Figure 10 is shown a simple cantilever consisting of two faces of equal thickness and a core. The end deflection of such a cantilever is calculated by the formula:

$$d = \frac{WL^3}{3EI}(1 - \mu_{12c}\mu_{21c})\left(1 + \frac{1}{4}\frac{h^2}{L^2}e\right)$$

where

$$2e = \frac{\beta_f}{\alpha_f}\left[1 - 3\left(\frac{c}{h}\right)^2 + 2\left(\frac{c}{h}\right)^3\right] + \frac{\beta_c}{\alpha_c}\left[3\left(\frac{c}{h}\right)^2 - 2\left(\frac{c}{h}\right)^3\right]$$
$$+ \frac{1}{\alpha_f G_f}\left[2 - 3\left(\frac{c}{h}\right) + \left(\frac{c}{h}\right)^3\right]$$
$$+ \frac{1}{\alpha_c G_c}\left[3r\left(\frac{c}{h}\right) - 3r\left(\frac{c}{h}\right)^3 + 2\left(\frac{c}{h}\right)^3\right]$$

and

$$\alpha = \frac{1}{E_1}(1 - \mu_{12}\mu_{21}) \qquad \beta = \frac{1}{E_1}(\mu_{12}\mu_{23} + \mu_{13})$$
$$r = \alpha_c/\alpha_f$$

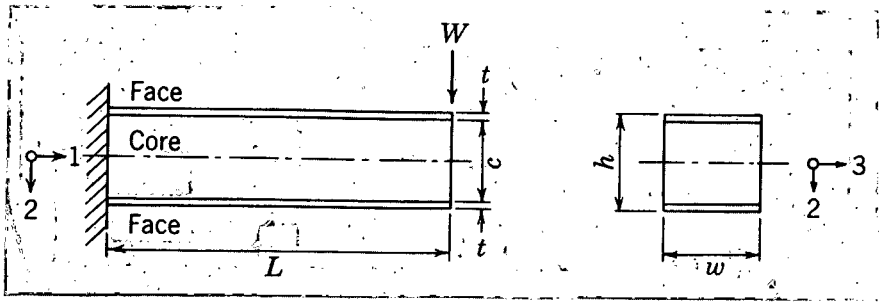


FIG. 10.—SANDWICH BEAM.

In most cases an excellent approximation may be had if the following simplified expressions are used:

$$d = \frac{WL^3}{3EI}\left(1 + \frac{1}{4}\frac{h^2}{L^2}e\right)$$

$$2e = \frac{E_f}{G_f}\left[2 - 3\left(\frac{c}{h}\right) + \left(\frac{c}{h}\right)^3\right] + \frac{E_c}{G_c}\left[3r\left(\frac{c}{h}\right) - 3r\left(\frac{c}{h}\right)^3 + 2\left(\frac{c}{h}\right)^3\right]$$

where

$$r = E_{1f}/E_{1c}$$

In the foregoing expression EI is stiffness factor or bending rigidity of the composite cross-section computed by taking the sum of the separate EI values of skins and core about the neutral axis.

μ_{12}, μ_{21}, μ_{23}, and μ_{13} are values of Poisson's ratio in the directions indicated. For isotropic materials these values would all be the same. The subscripts f and c refer to face and core materials.

Edgewise Compression

If a sandwich column is loaded axially it will fail or buckle as an elastic or Euler column if the length is sufficient to allow such buckling before the skins or faces can wrinkle. An exact solution is cumbersome but a close approximation is the following:

$$P_{cr} = \frac{G_c cw(P_1 + P_2 + P_c) + P_2\left[4P_c + \left(\frac{c}{c+t}\right)^2 P_1\right]}{P_c + \left(\frac{c}{c+t}\right)^2 P_1 + G_c cw}$$

where c is the thickness of the core, t that of each face, w the width of the column, and

$$P_1 = (\pi^2/2)(c+t)^2 twE_f/L^2$$

is the buckling load of a column whose faces are connected by a core perfectly rigid in shear, provided t^3 is considered negligibly small;

$$P_2 = (\pi^2/6)t^3 wE_f/L^2$$

is the buckling load of the two faces considered as independent columns (the core having no shearing rigidity);

$$P_c = (\pi^2/12)c^3 wE_c/L^2$$

is the buckling load of a column consisting of the core only; E_f, E_c, and G_c are Young's moduli for the faces and the core, respectively, and the shear modulus of the core; L is the distance between inflection points (equal to the length of the column if it is pin-ended).

In sandwich design an attempt is made to provide a core sufficiently rigid to provide enough support for the skins to prevent wrinkling or buckling of the skins until their ultimate tensile or compressive strength is reached. If this is not possible, then the design must be based upon the stress at which wrinkling of the skin may be expected, and the problem increases in complexity.

Acknowledgments

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DESIGN AND PROPER CARE OF ENGINEERING AND SURVEYING INSTRUMENTS

By Louis H. Berger*

(Presented at a meeting of the Surveying and Mapping Section of the Boston Society of Civil Engineers held on January 21, 1948.)

In the design of modern engineering and surveying instruments, as in that of many mechanisms, simplicity triumphs over complexity where practical service is of prime importance.

A recent tendency in the manufacture of such instruments, in America as well as abroad, has been the abandonment or the ignoring of designs characterized by simplicity, and the substitution of designs that are far more complicated. The claim is sometimes made that the complex designs insure accuracy and speed in the manipulation of the instrument. The fact is that the number of parts has been greatly increased in an attempt to accomplish in a different way what instruments of reputable make and simple design have always been capable of doing.

Some of these complicated designs have been announced as new improvements, but are they really improvements, or are they really new? The announcements may be innocently made, due to a lack of knowledge of that important element of technical as well as other education—history!

The basic principles of instrument design are a development of many years since, and several so-called new inventions can be traced back as far as a century and a half ago. Doubtless their re-introduction has been in many cases due to independent thought and action which might have been saved by a study of the annals of the instrument-making profession.

The most important question regarding these designs is, however, though they might once have been improvements, are they still superior in this state of mechanical history?

More and more is the fact realized that engineering and surveying instruments should be of simple construction, and free from complicated attachments since these are often a hindrance to the user instead of a help.

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In what are even in these days great distances, the time and expense involved in returning a complex instrument to the manufacturer would often be prohibitive, and many a nearby shop that might easily repair an instrument of simple design is not equipped with adequate auxiliary optical apparatus or trained men to make complete adjustment of the intricate parts of vital importance to the performance of the instrument.

Many an explorer who has equipped his party with over-elaborate instruments, and whose equipment has met with a mishap, has found himself seriously handicapped because of his remoteness to repair facilities. He has been able at the best to make only the crudest of emergency repairs.

It is advisable that the engineer or the surveyor should not only be able to adjust the instrument he is using but should also be able to make adequate emergency repairs. Every increase in complexity of design renders the instrument-user less able to adjust and to repair his instrument.

The engineer operating with a time penalty in his contract is in sore straits when he is forced to send an instrument to the factory for repairs because of minor disarrangement of delicate trains of prisms and gears.

Many treatises, mostly in leaflet or booklet form, treating of the care of engineering and surveying instruments have been prepared by university professors as well as by engineers and manufacturers. Some of these treatises have been authoritatively as well as excellently written and are hence of especial value. The subject is naturally one of varying ramifications, so that something of importance may often be learned from persons whose particular experience enables them to give advice or hints that are not common to all these treatises.

It is not the purpose of this paper to criticize or to add unduly to what has been written by experts in different lines of engineering and surveying work, but instead to emphasize some of the highlights, many of which have been already pointed out to some degree by various authorities. It is especially intended to indicate to what extent, other than in a recognizable emergency, an engineer should attempt that which should preferably be done by a factory expert.

The essential points in the manufacture of a strictly high-grade, properly functioning surveying instrument are as follows:

1. Materials of the highest quality.
2. Workmanship consistent with such materials.
3. The proper assembly of parts to constitute an accurate, dependable instrument and assure its long life.
4. Exacting final tests before delivery, simulating anticipated field conditions.

Like all other instruments of extraordinary precision, engineering and surveying instruments require handling of the highest degree of care. However, unlike other delicate devices, they are not only subjected to climatic and other weather conditions of the severest nature, but are accorded various kinds of especially rough treatment. Thus, in the design of such instruments the different conditions under which the finished product will in all probability be used must be intelligently anticipated.

Because of the exceptional and varied rough treatment that these instruments must naturally undergo, it is in the highest degree advisable that the user should be instructed as to the proper care of whatever instrument he is required to use, and the precautions he must take to avoid serious troubles.

The careful handling of surveying instruments is most naturally acquired through an understanding of their adequacy and a consequent respect for the necessary nicety of their manufacture. Such refinement of production not only includes intelligent design along sound structural lines, but the construction and fitting of parts to achieve proper tolerance under all possible conditions.

Proper specifications for surveying instruments call not only for an assurance of satisfactory stability, but also for the anticipation of any possible accident, so as to reduce resulting damage to a minimum.

It is the custom of many engineers and surveyors to return their instruments to the factory at least once every six months for inspection and proper cleaning and oiling. Such careful users are especially aware of the fact that when accidents have occurred, it is best to send the instrument to the maker for repairs and adjustments. While any manufacturer realizes the desirability of such practice, he is sympathetically aware of the oftentimes imperative necessity of emergency repairs in the field where time is an important factor. As a preventive to disaster on such occasions the maker is naturally especially solicitous that the user shall have instantly available knowledge of the proper care of his instrument.

When an emergency occurs, he should be sufficiently versed in the functions and interdependence of the various parts to make at least adequate emergency makeshift repairs. Through constant watchfulness regarding any possible need of adjustments and cleanings, the careful user of surveying instruments can reduce the likelihood of enforced repairs to a minimum.

The user of the instrument should learn to discriminate carefully between the adjustments and repairs which he can and ought to make himself and those which he should not attempt. Harm may be done by neglecting to make repairs; also, by trying to perform in the field a delicate operation which only the maker with the proper equipment should attempt.

Common sense is a safe guide in the handling of instruments. One of its dictums is, for instance, that in lifting a Transit or a Level it should be handled by the leveling base or foot plate—the structural supports designed to serve as the instrument's foundation. To lift an instrument by the telescope or under the horizontal circle is to take unwarranted liberties with what is really the heart of the instrument, which should certainly not be subjected to needless strains.

If it is necessary to lift the instruments by its superstructure, it is advisable to grasp the base of the standards near the horizontal plate, being careful during the lifting not to exert any lateral pressure.

Teachers of the art of surveying are well aware of the constant care and restrain that should attend the handling of an instrument. A student is taught, therefore, to exercise all necessary precautions in the use of his instrument. He is taught to tighten screws carefully and to try the operating parts with gentle pressure. If, through his initial ineptness, certain parts of the instrument fail to move in proper accord with other parts, the student is instructed to find out just why this happens, instead of, for instance, being permitted to exert his full strength, against the strength of the clamp. The student is instructed that he should be careful not only when using the instrument, but also when making the adjustments, when cleaning it, and when returning it to its box.

Sensitively careful treatment of a surveying instrument is necessary even in its leveling. It is possible through binding one of the screws of a four-screw leveling base to apply a pressure of several hundred pounds. It is easy to imagine the distortion which continual application of this pressure can produce on the socket of the leveling

head. What is even more injurious is that this pressure is transmitted as a direct thrust on to the vertical spindles of the instrument.

The respect for his instrument that is taught the student is reflected in the selection of tools and materials for use in its adjustment and cleaning. Only the finest grades of watch or clock oils should be used. Light machine oils, however purified they may be, are not suitable as lubricants. Oils of the penetrating or cutting type should not be allowed to be on the same bench as the instrument.

The best oils and lubricants are a prime factor in lowering the maintenance cost of instruments. Oils and greases formerly recommended by manufacturers for the lubrication of instrument parts have, in many instances, been discontinued owing to superior products now on the market. Certain excellent watch and clock oils made by different manufacturers work well at even 20 degrees below zero.

Tools such as screwdrivers, spanner wrenches and adjusting pins should be of reasonable fit to prevent the burring of screws. These tools are part of the equipment furnished with every Transit, so there is rarely need for substitute devices. Such makeshift implements as brads, wire nails and surveyor's tacks are very poor substitutes for standard tools, which should therefore be carefully and constantly guarded against loss.

Care should be exercised if long adjusting pins are used, because there is likelihood of the application of too strong pressure on the capstan-head screws. Through the use of the long pins the threads of the adjusting screws or the adjusting nuts are likely to be stripped. The pins should not fit the holes in the screws and nuts too snugly, as the adjustment of the instrument is likely to be disturbed because of the difficulty in their removal.

The telescope of surveying instruments as now constructed with interior focusing require very little care and adjustment. It is better to put up with any such annoyances as a small amount of dust on the cross hairs or on the surface of the lenses until the instrument can be sent to the maker. A properly constructed telescope is provided with small air vents to prevent the creation of a vacuum in one end of the telescope tube when the slide is being focused on different distances.

Lack of proper ventilation in the telescope tube causes the condensation known as "dew-drops" to gather on the cross hairs and on the surfaces of the various lenses. This condensation will gradually disappear when temperatures have been equalized.

The exposed lenses should be dusted with a scrupulously clean piece of linen, free from lint, applied with a light circular motion.

Unless experienced, one should not attempt to adjust or clean the cross and stadia wires of the telescope. It will be found that they seldom need adjusting or cleaning. An inexperienced person attempting this adjustment may change the telescope's line of collimation.

If an engineer is going into remote sections it is advisable that he be equipped with an extra diaphragm. However, should it be found to be necessary to clean the diaphragm wires this procedure is advised: With an erecting telescope, whether of the exterior or interior focusing type, first unscrew the object glass by turning it to the left. Before doing so, however, observe if there are any marks to show how far the cell is to be turned to place it in the correct position. If there are no marks, a small mark should be made parallel to the line of collimation which will extend from the edge of the cell to the collar. When the cell is put back, it must be turned until these marks coincide exactly. The focusing pinion of the slide should be removed, and then the focusing slide should be carefully drawn out.

To remove the diaphragm, or cross-hair ring, of an erecting Transit telescope, one should proceed as follows: First remove the telescope spirit level. Then remove the two capstan-head screws and their washers at the right and left of the cross ring; next, loosen the remaining screws to such an extent that the ring may be turned 90 degrees; then insert a long, sharp-pointed stick into the hole nearest the objective end of the telescope, so as to hold the diaphragm. Next remove the two remaining screws, and the diaphragm may be taken out on the stick.

With an inverting eyepiece, the diaphragm is more easily taken out through the eyepiece end of the tube. It is not necessary to detach the telescope level or to remove the object glass, the focusing pinion or the slide. First remove the two opposite screws having slots which hold the eyepiece in its mounting in the outer barrel of the telescope. Then slightly release all four diaphragm adjusting screws. Remove the entire eyepiece mounting. Then by taking out two opposite diaphragm screws and inserting the stick already described above the diaphragm can be removed as before stated. The cross-wires may be cleaned by using a small wedge-shaped piece of fine-texture tissue paper, moving it gently forward and back along the wires. This oper-

ation requires a very steady hand and the use of a reading glass to prevent cross-wire breakage.

When the manufacturer has selected the correct metals for a focusing slide and the metal for the barrel in which the slide travels, and the slide has been very painstakingly fitted, the slide or barrel never requires lubrication of any kind.

This is also true regarding the older types of instruments where the objective lens is moved in a slide. The final fitting of such slides is necessarily a hand job, as the slide should bear everywhere throughout its length and not solely on two or three collars or rings.

If oiled, the lubrication will harden in cold weather, so that the slide will move only under undue force. In warmer weather the oil will thin, causing a dust-bearing and fogging medium to be freed within the telescope barrel, which will be deposited on the cross-hairs and on the surfaces of the lenses.

The standards supporting the telescope should be very low, light, and stiff, with great lateral strength to resist any possible outside strain that may come upon them in rough usage. When their design is faithful along sound structural lines, the standards require but little attention. The bases of the A-shaped standards and the base of the yoke standard frame, both of which types are mounted on the horizontal vernier plate, should be sufficiently large to provide firm lateral support to the telescope. If not, trouble may develop in the trunnions or telescope axes.

If a fall occurs, it is usually one of the standards that suffers most. It should be remembered that when a standard becomes bent, the instrument maker is pre-eminently well equipped to straighten it properly.

The cylindrical trunnions of the telescope axis should be encased by end thrusts, which protect the bearings against dust, and also provide extra stiffness to the standards in case of a severe accident. If emergency repairs are attempted on the standards when they are damaged by a fall, their readjustment will generally be found to be arduous. If the bearings are of the cylindrical type, this adjustment will be less difficult than with bearings of other types. With Transit instruments equipped with cylindrical bearings, damage to the standards caused by a slight knock or blow are easily corrected, since the resulting error will be lateral.

The true form of the trunnions should be preserved and there-

fore they should be oiled frequently with the best grade of oil. There are now devices applied to the instrument which provide constant lubrication for the axis trunnions. A unique and effective plan is the use of small felt pads, which act as reservoirs serving to keep the axis bearings continuously lubricated.

The balance of a telescope, when equipped with an interior focusing slide (through the use of which the position of the object glass remains unchanged in relation to the cross-hairs, and the focusing for various distances is accomplished by moving an auxiliary lens), is manifestly superior to the balance of a telescope having a slide equipped for exterior focusing, through the use of which the object glass is moved away from or toward the cross-hairs to focus for various distances.

The question has often been asked: "Which is the better practice in order to preserve the balance of the telescope—to tighten the telescope slightly with a clamp, or to provide the axis trunions with brakes?" In either type of focusing slide, there is a lack of balance when the telescope is focused for short distances. Braking the trunnion axis to balance the telescope by forcing the metal caps of the standards down on the trunnions to produce friction may in time prove injurious to the trunnion axis through increasing the wear, particularly when the slide is focused for short distances.

A common method of braking the telescope is to tighten the telescope clamp to a very slight degree, and to tap the telescope into position with the finger. However, the only really satisfactory method is the one for which the instrument is designed; namely, to clamp the clamp-screw tightly, and use the tangent slow-motion screw to get the exact sight.

How to Take the Telescope Apart

The removal of an interior focusing slide calls for a different procedure from the removal of the exterior type. First remove the objective, then the focusing pinion, after which the telescope should be held in a vertical position so that the weight of the slide will force it to come out. The hand, or still better, some soft material such as a clean linen towel, should be held as a cushion under the slide to receive it and to prevent it from becoming damaged. In replacing the slide, one should be sure that the lens and the slide are perfectly clean. These suggestions are offered for use only in an emergency.

To remove the interior lens, or to make other necessary repairs to the interior focusing slide, requires the services of an experienced instrument maker, and should not be attempted in the field. Instead, the instrument should be returned to the manufacturer.

As most eyepieces of erecting and inverting telescopes now have protection guards for their respective slides, the correct procedure is as follows:

To take out the *erecting eyepiece,* first remove the dust cap, to which is attached a protection guard for the slide. Then remove the german-silver screw in the spiral slot to be found near the end of the main tube; take hold of the eyepiece and pull it out gently.

To take out the *inverting eyepiece,* first unscrew the eyepiece cap, unscrew the protection guard for its slide, remove the spiral slot screw, and then take hold of the eyepiece and pull it out gently.

In the instruction of students in surveying, it is naturally emphasized that it is necessary for a surveyor to be able to disassemble the parts of his instrument for cleaning and oiling, as well as to be able to make the proper field adjustments. In dealing with parts of instruments requiring attention, a clean bench and a dust-free room are, of course, recognized as prime requisites. An orderly arrangement of disassembling the parts is necessary for their uninterrupted and proper reassembly. Care should be taken that the graduated circle be placed in such a position that no dirt will possibly get on its surface.

When the parts needing attention have been disassembled, the first operation is to remove all traces of the old oil. The instrument should be thoroughly cleaned before any re-oiling is attempted. An absolutely clean chemical dissolvent that leaves no residue, such as naphtha or benzine that has not previously been used, should be employed. This should be applied with a piece of good-grade linen, thoroughly washed and absolutely free from lint. It is important to clean both the spindles and the sockets, or the purpose of the cleaning is defeated. The flanges of the spindles and their upper rests should be cleaned with another clean rag to remove all traces of film left by the dissolvent.

The proper way to oil the instrument centers is to apply a few drops to the spindles and their sockets, and their respective flanges. While no harm is done if the oil is applied liberally, as the excess oil will readily drain off, there is no sense in wasting the oil. The vernier plate clamp and the horizontal circle clamp should also be thoroughly cleaned and oiled, as well as the telescope clamp.

The leveling screws should be cleaned with naphtha or benzine, applied with a stiff clean typewriter brush. No heavy lubricants or greases should be applied, as dust will readily adhere to such substances and settle on the screws. Vaseline should not be applied to screw threads, as it is liable to harden, especially in winter, causing the screws to move very stiffly.

When the instrument is apart, the graduated circle may be lightly dusted with a fine camel's hair brush. No attempt should ever be made to clean the graduations even with abrasives so fine as optical rouge. Nor should silver polish, even of the highest grade, be used.

No Instrument Is Waterproof

Everything is done by the careful designer and builder of Transits to protect all revolving parts against water, grit and dust.

When instruments are to be exposed to particularly adverse conditions, such as in wet mines, additional precautions are necessary. No instrument can be made waterproof. For instance, it is practically impossible to make the instrument watertight around the periphery of the horizontal circle and still to insure adequate free motion of the instrument. Temperature changes alone would make this a practical impossibility.

Therefore, I repeat that no instrument can be made waterproof, which statement includes the telescope, the vertical and horizontal limbs and the compass. Everything possible is done to protect all revolving parts against water, grit and dust, especially when the instrument is to be used underground.

For several years, Transits used in wet mines have been furnished with the vertical circles and their verniers (whether of the flat face or edge graduated type) completely enveloped in casings or housings. This arrangement keeps the mine water from dripping on the graduations as much as it is possible to do so, but because the working parts must necessarily revolve it is not feasible to make them absolutely watertight.

Vernier cover glasses should, however, be fitted so that no water may enter when the instrument is in an upright position. Vernier glasses should not be embedded in cement or putty, as attempts to clean the glasses from below are apt to damage the vernier graduation. You are all aware of the fact that a vernier should not be removed, owing to the great difficulty in accurately recentering it so that it will register properly with the centering of the horizontal circle.

The graduations of the horizontal circle, after cleaning, should never be coated with a film of oil or vaseline, however thin, as any film over the graduations will be likely to cause erroneous angular readings because of overflowing of the oil or vaseline across the graduated lines.

In the servicing of an instrument, when it becomes necessary to remove the tarnish from an accurately cut graduation on sterling silver a coating of transparent lacquer *should not* be applied for reasons just given.

As a practical precaution, frequent cleanings seems to be the best preventive of instrument trouble. Of course an instrument should never be cleaned where dust or dirt is present.

Protection should be afforded instruments even when used in bright sunlight. The sensitive spirit levels with which Transits and Levels should be equipped ought to be protected from the direct rays of the sun whenever this is possible. A cloth hood should be provided in order to protect the instrument when it is not in actual use.

In regard to the practical simplicity which our firm has advocated and practised since 1871, I am pleased to note the recent increased demand for modern precise instruments instigated by the epoch-making activities in geodetic control surveys. This will of course be reflected in surveying instrument design, and in the wider use of the finer instruments as required in the geodetic control surveys. The new specifications will insure more accurate results than have hitherto been universally required.

This tendency toward increased precision must naturally be accompanied by an increased demand for the best instruments for their particular purposes. For instance, the Federal Agencies specify that instruments used by those local control surveys, such as city planning boards, whose work is under their supervision be equipped as herewith briefly outlined.

Instruments Equipped with a Three-Screw Leveling Base and Those Equipped with a Four-Screw Leveling Base

A Theodolite with verniers reading to 10 seconds should have a three-screw leveling base instead of a four-screw base. The vertical centers, or spindles, of such instruments should not be made of the bronzes, but of tool steel, etc. (not hardened or tempered), revolving in sockets of soft gray iron, as the co-efficients of expansion are less

susceptible to temperatures changes that affect the accuracy of instruments of a higher order.

A Precise Tilting Dumpy Level, such as is employed in first and second-order leveling, with an independent micrometer screw which is used for elevating and depressing the telescope, and to which telescope a very sensitive spirit level is attached, should also be equipped with a three-screw leveling base instead of a four-screw base. An instrument having a three-screw base, owing to the greater radius of the leveling screws, promotes far more stability of the superstructure of the instrument, particularly in a high wind.

Besides this, an instrument having a three-screw leveling base is much more quickly and evenly leveled up, and there can be no strain brought about by the pressure of the leveling screws when an instrument is so equipped, especially when it is mounted on a suitable tripod having a head of large diameter, with cheek pads of great width where the bolt hole for the leg is located.

For general engineering practice the compact instruments equipped with a four-screw leveling base will in all probability always constitute the bulk of instruments manufactured in America.

Let us now turn our attention to a brief consideration of the latest trends in materials used for surveying instruments.

Aluminum and Its Uses for Engineering and Surveying Instruments

Although leading manufacturers have built many instruments of special design for special purposes, for various governments, some with parts of aluminum and its alloys, while other essential parts were of bronze, steel, cast iron, etc. (in order to give the necessary wearing quality and accuracy called for in a satisfactorily working instrument), the responsibility for proper functioning of such instruments made with parts of aluminum was naturally that of the parties who drew up the specifications.

I wish to emphasize that these alloys, when used for field instruments, must be used with extreme caution.

An instrument, in order to have stability in high winds, and when otherwise subjected to vibration, should have considerable weight, as nothing can be achieved by the engineer under such conditions if his instrument is too light, which will be the case if most of the parts of which it is constructed are of aluminum.

Aluminum has always been considered by us not to be the proper material of which to construct engineering and surveying instruments.

Instrument parts made of aluminum should not revolve or travel in other parts made of aluminum as they wear out very quickly since this metal flakes and causes parts to become frozen or fretted.

Our comparison between the use of aluminum and the other well-known metals applies to the following parts:

The focusing slides for the object glass and the eyepiece.

The telescope barrel and the horizontal axis trunnions and clamp.

The vertical circle and its guard.

The standards, both A- and U-shaped.

The horizontal circle and the vernier plate.

The centers, both inner and outer, and the leveling head.

The center clamps for both upper and lower plates.

The foot plate and the tripod head.

The instrument parts, some of which have just been listed, should, when made of aluminum, never revolve or move in parts of bronze, cast iron or steel.

That stable metal, bronze—time tested and proved to be the best material for the construction of the instruments—has reached a high degree of perfection through recent advances in metallurgy.

The Proper Material to Use on Which the Graduations Are to Be Cut for Vertical and Horizontal Circles and Their Verniers

Different metals on which the graduations are cut for horizontal and vertical circles and their verniers are as follows: Nickel silver (commonly known as german silver), brass, bronze, aluminum, solid silver, and sterling silver. Nickel silver, brass and bronze are altogether too hard on which to cut accurately spaced graduations, as with any of these metals it is difficult to obtain the proper width, evenness and depth of line. On the other hand, solid silver is much too soft to use because the edges of the graduations where the reading of the vernier is made can become so easily damaged, this being brought about by too frequent cleaning of the graduations.

Of late years aluminum has been sometimes selected as a material on which to graduate, because it is lighter in weight than other metals used for this purpose and because it is said to be more free from the effect of tarnishing than is solid or sterling silver. Graduations

produced on brass or bronze (which metals are later silvered and coated with a transparent lacquer to give the lines a light back-background), as well as graduations cut on aluminum, are usually found to be quite ragged and jagged in appearance when viewed under a microscope; such lines are not conducive to accuracy.

With proper Automatic Dividing Engines, accurate graduations are best produced on sterling silver (925/1000 fine) which is a metal neither too hard nor too soft. The reading edge of the graduation is less likely to become damaged than would be the case where solid silver used. An accurate graduation should show lines which have been cut V-shaped, with parallel width for their entire length and should have great depth and sharpness of line.

To substantiate my remarks on the use of a proper metal for graduations, I wish to cite an instance which occurred several months ago. One of our Federal Departments that purchases instruments in large quantities specified in its bid forms that manufacturers would have to graduate the vertical arcs and verniers on nickel silver. Since then, other specifications have been received from them stating that the graduations were to be on aluminum. However, their most recent specifications call for all graduations to be cut on thick hard-rolled sterling silver.

Sensitive Spirit Level Vials Better Than Those of the Insensitive Type for Transits, Theodolites and Leveling Instruments

Transits, Theodolites and Leveling Instruments should not be equipped with insensitive spirit levels. Such spirit level vials are those usually found mounted on the vernier plates and on the A- and U-shaped standards of Transits and Theodolites, and the level vials which are mounted under the telescopes of Transits, Theodolites and Leveling Instruments. One of the reasons that insensitive spirit levels have been chosen is apparently to hide defective workmanship, such as eccentricity of the compound vertical centers, or spindles, of Transits and Theodolites; also of the spindles of Leveling Instruments. The use of insensitive spirit levels is not recommended on instruments when the accuracy demanded by present-day survey work is taken into consideration. It is not unusual to discover a Transit or Theodolite the plate levels of which have a value of only two minutes of arc, even though the horizontal verniers of the Transit may read

to 20 seconds; also Theodolites with verniers reading to 10 seconds. May I repeat that in my opinion instruments equipped with insensitive spirit levels should never be employed when accurate surveys are to be made such as those advocated by the United States Coast and Geodetic Survey.

Therefore, my advice to surveyors is as follows: See to it that your instruments are equipped with spirit levels having at least sufficient sensitivity to be consistent with the powers of the telescope and the fineness of the graduated limbs and verniers; and when level vials need replacement be sure that the servicing department furnishes you with new vials of the sensitivity recommended as follows:

When the A and B horizontal verniers of a Transit read to a single minute, or even to thirty or twenty seconds, be the size of the instrument 6¼, 5½ or 4½ inches at the edge of graduation where the reading is made, the plate levels should have a sensitivity of at least 60 seconds of arc measure for each 2-millimeter division of the graduated scale etched upon the outside of the vial.

The telescope vial for an ordinary Transit, Wye or Dumpy Level can be either 30 seconds or 20 seconds, depending upon the sensitivity required by the engineer.

A 7-inch or 8-inch Theodolite, with A and B verniers reading to 10 seconds, should have both plate level vials with a sensitivity of 45 seconds for each 2-millimeter division. When a telescope vial is to be added to this type of instrument, it can have a sensitivity of either 20 seconds or 30 seconds.

Filling Fluids for Spirit Levels

The inner surface of a spirit level should of necessity be ground to a constant radius of arc; it should be barrel-shaped. The air bubble, or blister, of the vial should move very smoothly without hitching for any part of its traveling distance. It should *not be filled* with a slow-traveling fluid, such as wood alcohol, synthetic alcohol, or even grain alcohol. The level should be filled only with a mixture of two-thirds high-proof filtered grain alcohol and one-third filtered ether, the best obtainable, since cheap commercial ether will attack the truly ground curve of the inner surface of the vial, causing pitting which will obstruct the flow of the blister and will result in inaccuracies after the instrument has been leveled up.

Instruments for use in the Arctics should have spirit level vials

filled with the purest ether as it is the only non-freezing fluid suitable for use in extremely cold climates.

No Lubrication Required on Telescope Focusing Slides

I do not believe that the main focusing slides of Transits, Theodolites, Leveling Instruments and Alidades, whether of the exterior or interior type of telescope should be lubricated with watch or clock oils, grease or talc. When slides are properly fitted there should be no looseness whatsoever and for this reason there can be no room for oils of any kind. We have not found it necessary to use any lubricant in the slides as made by us. In a poorly fitting slide that will permit the use of oil, there will be either too much or too little oil according to the position of the telescope when sighting on different distances. The weight of the slide will cause displacement of the lubricant. Poorly fitting slides affect the line of collimation, whether the telescope is of the exterior or of the interior type. In hot weather, when a summer oil is used, there is danger of the oil's finding its way to either or both the cross-wires and lenses. The lenses exposed to oil lose their sharpness of definition. When winter oil is used it prevents free motion of the slide.

If the focusing slide frets, a little fine watch oil may be used to grease the slides as an emergency practice, but the instrument should not be used more than necessary before being repaired, in order to prevent this fretting condition from becoming worse.

Protection of the Instrument

1. Avoid falls and accidents, liable to happen if instruments are left standing unguarded while not in actual use, in a street, road, pasture, etc., or in close vicinity to a concrete-mixing machine, or where blasting is done, in mines, etc.

2. Avoid exposure of the instrument to excessive heat, or to dampness, rain or dust, by using an umbrella, or by a bag thrown over it when not in use.

3. Avoid carrying an instrument from a cold atmosphere into a warm room without the protection of its box or a bag thrown over it, as the vapor thus condensed on the metal surfaces will settle a film on exposed graduations, making them indistinct and difficult to read.

4. Avoid neglect of protecting the lenses of the telescope from

moisture, dust or rain, by the cap and eyepiece lid provided for them as any scratch or film collecting on or between them will greatly impair the clearness of sight through the telescope.

5. Avoid carrying an instrument on its tripod or in its box without slightly clamping its principal motions, so as to prevent unnecessary wear of centers, etc. When carrying on the tripod, clamp the telescope in a Transit when placed in line with its centers; clamp the telescope in a Level when it is hanging down.

6. It should also be remembered that perspiration from the hands will injure the lenses and the silver surfaces of the graduations. Therefore, avoid touching these parts with the fingers.

Extra Parts and Utensils Necessary for Engineers Going to Remote Places

The engineer going to remote places should equip himself with extra wire diaphragms, extra spirit level vials mounted in their casings, extra tangent and leveling screws, and extra clamp screws. He should also equip himself with proper lubricants, an assortment of adjusting pins—large and small, clean linen rags, and long tapered center sticks for cleaning the sockets in which the centers revolve. The cost of these extras is slight when compared with the loss of time, inconvenience or inability to procure them from sources other than the factory.

Emergency Field Repairs and Adjustments to Be Performed Only If Absolutely Necessary Due to Isolation, Time Factor, Etc.

Care of the Lenses

Much cleaning of the lenses must eventually cause dimness of the image and a loss of definition. It is better to avoid the necessity for frequent cleaning by protecting the lenses as much as possible from exposure to dust and dampness. If the cap is placed over the object glass and the eyepiece cover is kept closed except when the instrument is in actual use the lenses will last much longer.

Care of the Object Glass Slide

The object glass slide should work smoothly but not too freely. It should not be so loose that it will drop when the telescope is turned to a vertical position. The object glass itself should not be greased or

oiled, but the rack and pinion motion should be greased with rendered marrow. In cold climates the marrow may harden so as to cause difficulty; in this case vaseline may be used to advantage, but must be renewed frequently in cold weather.

If the focusing slide seems to work too hard, everything else being right, this is generally caused by the hardening of the lubricant on the pinion. In hot weather the same cause may make the focusing slide work too freely by softening, so that the slide will not stay in place when in a vertical position. If the slide moves too freely it should be tightned by running out the slide to its full length and then applying a screwdriver to the screw on top of the focusing screw, and turning very slightly until the required friction is obtained. If the slide works too tightly, run in the slide, unscrew the top screw one turn, gently tap it by the screwdriver handle to release it, and then tighten to the required stiffness.

Care of the Cross-Hairs

If dust is allowed to settle on the cross-hairs, they will present a rough, irregular appearance which is troublesome. To remove this, unscrew both the object glass and the eyepiece and gently blow through the telescope tube. Before putting the lenses back into place cover both ends of the tube with a cloth and wait a few minutes for the moisture to evaporate. The object glass should be screwed up well against its shoulder. As the cross-hair adjustment may have been disturbed by the removal of the object glass, this should be tested after the lenses are put back into place. Do not unscrew the object glass unnecessarily, for this is likely to alter the collimation adjustment.

Care of the Magnetic Compass Needle

The compass needle rests on a finely sharpened pivot, and the accuracy of the results obtained with the needle depends more upon keeping the pivot point sharp than upon any other condition. Therefore, the needle should be let down as gently as possible on to its pivot. When not in actual use it should always be raised by means of the lifter. If, when lowering the needle, it is found that it swings violently to reach its north and south position, check this movement with the lifter to prevent undue wear on the pivot. When the instrument is being carried the needle should, of course, be lifted against the glass cover.

The preservation of the magnetism is also an important factor in the proper action of the needle. Magnets, or iron or steel objects, should not be brought close to the needle on account of possible permanent injury. When taking readings of the needle, care should be taken that no iron carried on the person is brought near enough to affect the direction of the needle.

In cleaning the glass cover of the compass box a silk handkerchief should not be used on account of the likelihood of its electrifying the glass. After cleaning the glass, breathing upon it tends to draw off the electric charge. If the needle is attracted to the glass when a reading is being taken the cover may be touched with the moistened finger, when the charge will at once be drawn off.

When the instrument is sent from the factory the needle is balanced for our latitude by means of a fine wire. When used in a different latitude the position of the wire may have to be changed. If the needle is but little out of balance it is better not to disturb it. If adjustment is necessary, however, raise the cover glass by first prying up with a knife-blade the circular split ring which holds it (applying the blade where the ends of the ring meet), then lift the glass by means of a piece of softened beeswax. When the construction of the compass box is different from the one just outlined, unscrew the knurled threaded bezel ring which holds the cover glass of the compass. The needle should then be raised by means of the lifter and removed with a pair of tweezers. Move the wire counterpoise in the direction required to balance it, replace it on the lifter, lower the glass and then test the balance by letting the needle down gently on to the pivot. If it becomes necessary to remove the glass entirely, it may be done by unfastening the screw that holds the stud for the telescope tangent screw to the standard.

Care of the Graduations

To remove dust from the graduations and verniers use a fine camel's hair brush. Do not rub the graduation, especially near its edge, as the slightest wear on this edge will make it difficult to read angles accurately. If it becomes necessary to clean the graduations the greatest care must be taken not to scratch the silver surface nor to injure the edge. To clean the silver graduations apply some fine watch oil and let it remain for several hours. Then take a piece of soft (old) linen and rub lightly until dry, but without touching the

edge of the graduation. If stains still remain, moisten the finger with vaseline and apply it to the surface; then wipe the finger dry and rub it once or twice around the graduation. This should not be done, however, unless absolutely necessary, because it is likely to destroy the fineness of the graduation. See that the inner edges of the graduations and verniers are free from dirt and grease before replacing the inner center. The vernier should not be unscrewed from the upper plate nor the centering of the outer center altered, as their adjustment is too delicate for anyone but a maker.

The glass covers protecting the verniers, as well as the compass cover glasses, need to be carefully brushed and cleaned the same as the lenses, because any scratches or films will impair their transparency. If the reflector shades for the verniers become dirty they should be taken from their frames and washed in soap and water. They should be kept clean so that they will illuminate the verniers and the graduations. The cleaning of the graduations must take place after the lower portion of the instrument has been taken apart for cleaning of the centers, a description of which I shall now give.

Care of the Centers

The two vertical axes about which the circle and the vernier plate turn, or *centers,* as they are called by the maker, form one of the most important parts of the instrument. They should revolve freely but without the slightest looseness. If the centers do not revolve freely they should be cleaned as soon as possible.

To do this, first unscrew the milled-head nut at the end of the cylindrical tube which is opposite the upper tangent screw. This should be done cautiously or the spiral spring opposing the tangent screw may fly out and become lost. Then unscrew a small cylindrical cup which carries the plumb line hook and which is at the bottom of the centers. After unscrewing the nut attached to the inner center a gentle pressure upwards will lift the vernier plate out from the lower part of the instrument. Take a stick having about the same taper as the inner center, wrap some soft linen slightly soaked in benzine or naphtha around it, and clean the inside of the socket carefully. Then remove this piece of linen and wrap a fresh dry piece of linen around the stick. The centers and their flanges should be treated in a similar manner. Before applying the fresh pure watch oil, care should be taken that all dust or other foreign matter is removed from the sockets,

the centers, and the graduations. When these surfaces are perfectly clean, fresh oil should be well distributed on all the bearing parts. It would be well to examine also the bearing surfaces of the vernier plate clamp, the lower clamp, and the telescope axis clamp, and clean thoroughly if necessary. After the instrument is thoroughly cleaned and oiled and the nuts and springs replaced, the instrument should turn freely and yield at the slightest touch of the hand.

Great care should be used when removing or replacing the inner centers that nothing injure the edges of the graduated circle or verniers. When the plates are together the vernier and the circle should revolve in the same plane. The space between the two should have the appearance of a uniform fine black line.

Care of the Leveling Screws and Tangent Screws

The screw threads of leveling or tangent screws should be kept clean to avoid sticking. For this purpose use a stiff toothbrush to clean off the dust, then oil the threads and work them in and out. Continue alternately brushing and oiling until they work smoothly. Do not leave oil on the threads, as this would merely collect dust and soon make matters worse.

Care of the Tripod

The tripod legs should not be allowed to become loose. The wing nuts and bolts at the head of the tripod should be well tightened up against the wood. If one of the tripod legs is raised and allowed to fall of its own weight it should sink slowly to the ground. If it drops quickly it is too loose; if it does not fall it is too tight. The shoes should be examined to see if they are loose. The screws in the shoes should be drawn up tightly. The points of the shoes should be sharpened whenever necessary. If these matters are not attended to the instrument may be unsteady. The tripod should be kept well varnished to preserve the wood from the effect of moisture, and should be wiped off if wet.

Care of the Instrument in the Box

The safety of the instrument depends upon the way it is packed in the box. If any of the wooden blocks should become loose they should be repaired at once; the rubber cushions underneath the box, the leather straps, buckles, hinges and locks should be examined fre-

quently and kept in good condition. The box should be varnished whenever necessary. If it becomes wet it should be wiped dry.

Do not place the instrument in the box without having first brought the four leveling screws to bear against the foot plate, and make sure that the telescope clamp and the clamps for the upper and lower plates are tightened. Be sure that no part of the instrument strikes the side of the box or the door when closed.

Care of Instruments Under Low-Temperature Conditions

For many years we have helped engineers and explorers in their selection of instruments for use in the frigid zones. For this reason I think it well to include some hints in regard to obtaining the best results from your instruments in the event that you are called away from a temperate zone to places such as Greenland, Iceland, or other bases at which United States Army engineers and troops are now stationed.

When you requisition a Transit, Theodolite or Leveling Instrument, make sure that only the proper metals have entered into the construction of the instruments you are about to purchase; for otherwise your survey work may be interrupted because of the metals used not being properly related to one another, thus causing friction, fretting and freezing of the principal sliding and revolving parts which go to make up these instruments. It must be borne in mind that you may be hundreds or thousands of miles from an instrument maker's or repairer's shop.

Aluminum should not be used for sliding or revolving instrument parts. Neither should a spindle of steel be allowed to revolve in a socket of bronze, because the coefficients of expansion of the two metals are not alike. A bell-metal inner center for a Transit, a Theodolite or a Level should revolve in a socket of bronze. There is no better combination of metals (provided the instrument is *not equipped* with a compass) than a steel center revolving in a socket of close-grained cast iron, because steel and cast iron have about the same coefficient of expansion.

Before using instrumental equipment in the Arctic and other frigid regions, it is necessary to see that all lubricating oils and greases are removed from such parts as the telescope focusing slides, the focusing pinion and the rack in which the teeth of this pinion engage, the centers and the sockets in which they revolve, the clamps,

and the leveling, tangent and clamp-screw threads, for otherwise the engineer cannot manipulate his instrument. As you can well realize, the greases and oils used for lubricating these parts would cause the instrument to freeze. Therefore, for best results lubrication should not be used. Herein is another reason why it is essential to know something about the metals which enter into the construction of your instrument, since the slides, the center, the sockets and all other moving parts should work freely and without looseness when no lubricant can be used.

Several Arctic explorers with whom I have conversed have told me that they always leave their instrument equipment cached in a snowbank, never taking the instruments into their huts or igloos. This is in order to avoid moisture collecting on the lenses of the telescope and the graduations, this condition being brought about by the difference between the indoor and outdoor temperatures.

Some explorers believe that it is best to work as long as possible without gloves. It is for this reason that we have often been asked, when supplying instruments for use in the frigid zones, to equip them with celluloid prongs jutting from the periphery of the heads of leveling, tangent and clamp screws; or to make the heads of these screws, and also the heads of the focusing pinion, as well as the knurled periphery of the eyepiece cap, of celluloid, hard rubber or bakelite, so that the engineer who finds it necessary to work with bare hands will not find his fingers freezing to the metal surfaces of instrument parts not so made.

The general exterior finish of the instrument also needs attention along similar lines. Instruments for Arctic and other low-temperature use should be leatherized or cloth-finished.

It would be well to take along extra spirit-level vials filled with the purest ether, as it is the only non-freezing fluid suitable for use in extremely cold climates. It is also important that the length of the bubble of the vial be correct for low temperatures, as a short level vial bubble at a temperature of 68 degrees will expand considerably when used in the frigid regions.

It cannot be emphasized too strongly that a surveyor in the frigid zones should be exceedingly careful as to the suitability of his instruments for efficient use in a frigid climate. If, as I have suggested, the maker has not taken the necessary precautions in the selection of the metals that enter into the construction of his instru-

ments, using only metals that are properly related to each other, having approximately the same coefficient of expansion; if he fails to see that they are practically free from zinc, that causes fretting of the closely fitting revolving and sliding parts, so that the parts move freely without lubrication of any kind, then, because of such negligence on the part of the maker, the surveyor with instruments made valueless by their inadaptability to the conditions that confront him, is, indeed, in a serious plight.

It is on record that the failure of certain Arctic expeditions was due to the worthlessness, under severe climatic conditions, of the surveying instruments employed, which failed to function properly because of the improper relationship of the metals used in their construction.

I esteem it a privilege to convey to you accounts of the experiences of prominent users of modern engineering and surveying instruments under especially trying climatic conditions.

Survey Work in the Arctic

The following notes are by Bradford Washburn, Director of the New England Museum of Natural History, Boston, Mass.

"In carrying surveying instruments from camp to camp on Arctic expeditions it is extremely important to remember that the experienced surveyor will not himself always be in charge of the instrument, and that it should therefore be packed in such a way that it can be handled with relative disregard without incurring damage. This refers particularly to having some sturdy point of contact well above the base of the instrument, which is often the only point of attachment in the standard box. Our experience in carrying instruments on the Alaska-Canada boundary has shown that they are most easily transported on the Yukon packboard, which is light, and to which the instrument can be very easily attached. In moving from one camp to another we carried our instruments on dog sledges where the going was easy and by back-packing in difficult terrain.

"I feel that every effort should be made to use a tripod that is particularly versatile. In other words, an attachment should be provided to the cameras used by the expedition so that they can be attached to the surveying tripod. This is probably impossible in the case of the plane table, which necessitates having a Johnson head, but we have frequently and successfully used a single tripod for our transit, our panoramic camera and our moving picture camera, thus saving a vast amount of duplication and extra baggage.

"In setting up the transit in country deeply covered by snow or ice in excessively cold weather, we almost invariably dug a pit, roughly six feet in diameter and four feet deep, throwing the snow up to make a wall around it

for our astronomic observations. A small embayment was cut out of one side of the pit for the recorder to sit in.

"It is extremely difficult to seat a tripod firmly in any kind of soft, or even wind-packed, snow. It has often been suggested that the tripod be equipped with ski-pole rings attached about six inches above the bottom of the legs. We have twice experimented with such equipment in Alaska and found it worthless. The tripod does not settle into the snow, but the springiness of this arrangement makes the entire combination valueless from a surveyor's standpoint.

"The only solution in loose snow is to pack the area firmly around each leg by jumping on the snow without snowshoes on. When this is completed the tripod can be set up, and water prepared from snow heated on the Primus stove can be poured into a little pile of snow stacked around the bottom of each leg. The slush thus created can be tamped about the legs. It will freeze almost instantly, and if sufficient slush has been made around each leg the tripod will stand very squarely. This was the only way in which we ever succeeded in getting our rounds of angles to close accurately when pitched on snow or ice. When we did not use this technique, our rounds of angles were often as much as five or seven minutes in error in a single circuit of 360 degrees.

"It is needless to say that an ice-axe must be carried by the party in order to chip the frozen slush from the bottom of each one of the tripod legs at the conclusions of observations.

"The small Primus stove proved very handy for use in field work, whether or not the observer was able to be protected in a snow pit from the cold evening down-glacier wind (which was invariably present in clear weather). We did our best to protect the recorder by building some sort of a snow wall about him, and with our small stove we brewed a pot of tea or coffee that both observer and recorder could sip from time to time. I feel that there is no better way to keep one's fingertips warm than to keep one's stomach full of hot liquid. Whenever possible this stove was placed in the same pit with the recorder, between his feet so that as he sat doing his work the warm air above the pot would serve to warm his legs, to warm his angle book and to warm his fingertips when not writing. It is particularly valuable to have the surface of the angle book itself warm. As an individual I would much prefer to work with cold hands on a warm paper surface than to work with gloves on a cold angle book. Wherever possible the recorder used suede gloves lined with squirrel fur. The observer did likewise. Canvas mittens with changeable woolen liners reinforced with dry-tanned leather palms were always used for every other purpose. It is particularly important to have all leather used for any purpose in the Arctic dry tanned; it will otherwise freeze as stiff as rock. An airplane helmet is in many ways much more practical to use than either the parka or the heavy fur hat, inasmuch as its neat skin-tight fit makes it much less apt to come in contact with the instrument, even when the eye is virtually touching the telescope.

"Our instruments have never been oiled. We have either used kerosene or, preferably, Fairchild non-freezing camera oil for lubrication. This Fairchild oil provides excellent thin lubrication down to at least forty-below zero without binding, and has always been used most successfully to lubricate the shutters

of aerial cameras in excessive sub-zero temperatures. It is needless to say that whether we are discussing cameras or surveying instruments, it is most important to insist that any machine parts which must operate in contact with each other with close tolerances should never be made of metals with greatly varying coefficients of expansion.

"In conclusion it is also important to remark that in so far as possible all surveying equipment should be kept outside the tents occupied by the party, and they should never under any circumstances be taken into the cook tent."

Commenting on his experience in the Arctic, Mr. Anthony Fiala, the Director of the Fiala-Ziegler Polar Expedition, informed me that the members of his expedition wore heavy fur mittens or triple woolen mittens with canvas covers. For use with instruments, silk gloves were worn directly on the skin, under the mittens; but even with the gloves they could keep their hands out of the heavy mittens only a few seconds.

The instrument man was careful not to breathe on the exposed surfaces of the vertical circle and its verniers, nor on the glasses which protect the horizontal circle and vernier graduations, for if these precautions had not been taken the vapor from the observer's breath would have congealed instantly and formed a sheet of ice on the exposed surfaces.

Mr. Fiala emphasized the fact that in extremely low temperatures a transit, a camera, a rifle, or in fact any instrument in constant use, is kept in the open. To return the instrument to a warm surrounding would cause moisture to condense on the cold metal surface.

Another important point that he brought out was the necessity for removing all oil from working parts before exposing the instrument to freezing atmosphere.

The following suggestions have been given me by F. H. Peters, Surveyor General and Chief of the Hydrographic Service of Ottawa, Ontario, Canada:

"Regarding the set-up of the instrument, much would depend upon the type of work on which the surveyor was engaged. For ordinary stadia traverse work it is customary to merely trample the snow well and set up the tripod on the snow, pressing the legs down until satisfactory stability is obtained. The interval of time between reading of bearings on the backward and forward legs of the traverse or in measuring angles should, of course, be reduced to a minimum. Some dislevelment of the instrument would no doubt take place, and if the day was mild this might be considerable, but in work of this kind it would hardly be practicable to take the time required to attain a very stable set-up.

"For astronomical observation work, however, a good set-up is essential and the care which it is necessary to take depends upon the precision required of the observations. For our work where astronomical observations are used merely to control stadia traverses or to provide control for mapping from aerial photographs, we do not require the highest degree of precision, say, about a minute in azimuth work and about four or five chains in latitude and longitude. For this degree of precision we do not dispense with the instrument tripod, but our surveyors are instructed that a very solid set-up is essential. This may be effected by shovelling snow down to the bare ground. If the ground is at all soft, wooden posts have to be driven into the ground, but if the ground is reasonably solid this is not necessary. A set-up of this kind should limit the dislevelment of the instrument to not more than, say, 15 or 20 seconds during the two or three hours necessary for a latitude and longitude position.

"For precise work, of course, with astronomical instruments, a concrete pillar would be necessary.

"Regarding protection for the observer, this is purely an individual matter that the observer decides for himself. He would of course choose his observation point in a sheltered spot and take such other steps as would minimize his discomforts. We have found, however, that excessive condensation and frosting on the instrument frequently occur on night work and this can be very much reduced by having an observing tent designed for the particular type of observation required. We have used two types of tent—one for observations in the meridian and one for observations at a constant altitude of 45 to 60 degrees. The observing tent will provide protection for the observer, but it is pretty essential to have another tent, with heat, at a proper distance from the observing tent where the observer can thaw out between stars.

"As to whether an explorer uses gloves, this again is a personal question. The most frequent custom is to have thin wool or silk gloves or fingerless mittens for use on the instrument, with suitable warm mittens attached to the observer's clothing or hanging around his neck, into which he can plunge his hands when not actually working on the instrument. Some surveyors can work with gloves, while others feel that some of their fingers must be free to attain the proper instrument 'touch'. The design of the instrument, with large or small clamps and tangent screws, has some bearing on this matter. It is of course very desirable to have the metallic parts covered with some insulating material, and if they are not so covered a little surgeon's tape will provide a satisfactory covering.

"Regarding lubrication we have found, in general, that powdered graphite makes the best lubricant for very low temperatures. If no prepared graphite powder is available, the fine scrapings from a hard lead pencil will be satisfactory. The instrument will not move quite so freely as when properly lubricated at milder temperatures and this must detract somewhat from the precision, but it is the most generally used lubricant for temperatures well below zero.

"I understand that there are now on the market several types of oil which are advertised for extremely low temperatures. We have had experience with only one of these, which we tried out about four years ago. I give the surveyor's report as follows:

" 'When the instrument was set up at the commencement of the survey at a temperature of —38 Fahrenheit, it was found to be so stiff as to be unsuitable for use. On bringing it into the tent it immediately loosened up; the instrument was dismantled, all lubrication removed and it was relubricated with a mixture of coal oil and watch oil, after which no further trouble was experienced throughout the season.'

"When not in use, it is customary for surveyors to leave their instruments outside the tent, the principle being to avoid as far as possible any rapid change of temperature.

"We quite appreciate the importance of the differential coefficients of expansion of the metals used in instrument construction where temperature fluctuations are extreme. It recalls to mind an occasion when a surveyor came across his leveller and rodman employing a considerable amount of brute force to persuade the level to rotate. They said this was frequently necessary in the cold weather despite the fact that the instrument had been dismantled, carefully cleaned, and lubricated with graphite.

"One point which we have found desirable in winter work is to have the spider web diaphragm replaced by a glass diaphragm, as some surveyors maintain that the spider webs often break or buckle with extreme changes of temperature.

"I recall that one of our surveyors returned from a winter survey with evidence of bad surveying practice on his face in the form of a large brown burn on his left cheek. In northern latitudes the altitude of Polaris usually makes it necessary to change to a Diagonal eyepiece. This surveyor, however, had an india rubber neck sufficiently pliable to get his eye under the direct eyepiece and tried to avoid frozen fingers incurred in changing eyepieces, but it meant that when bisecting Polaris his cheek rested against the horizontal circle. Perhaps he valued his fingers more than his face."

L. O. R. Dozois, D.L.S., of the Geodetic Service of Canada, who has had considerable field experience in winter operations, has given me the following notes:

"It might be pertinent first of all to stress the fact that, generally speaking, a leveller has to manipulate his instrument during actual work very much more than is the case with a transit man.

"When a transit has been set up and the line established the observer not infrequently has considerable leisure while waiting on the axemen, chainmen or monumenting party and he will consequently be able to protect himself from physical suffering due to severe cold. On the other hand when a leveller, working with two rodmen, pauses to fight off frost bite or benumbing cold, the progress of the work is immediately affected.

"Another consideration is that while a stiffness in the vertical axis might seriously affect the measurement of angles, such a condition in a levelling instrument need not necessarily give rise to any error. This does not, however, imply that a freely revolving vertical axis is not desirable in a level but it is intended rather to accentuate the fact that each type of instrument has its individual reactions to low-temperature conditions.

"As regards the level, the following remarks are based on the experience of three winter seasons in unsettled northland.

"*Lubrication:* The vertical axis of the cone type has developed binding in only moderately low temperatures, say 0° Fahrenheit, when a film of oil is present. This has been rectified by wiping away oil and applying graphite by merely rubbing with a hard lead pencil, 2H to 4H.

"Although trouble of this nature was not found to be entirely absent in the cylindrical type of axis as incorporated in the design of the level used, it may be said that binding has not appeared until temperatures of about 20 degrees below zero prevailed.

"Difficulty arising in the focusing mechanism due to cold has not been encountered under conditions as low as 44 degrees below zero Fahrenheit.

"*Vial.* A levelling instrument for use in temperatures lower than 15 degrees below zero Fahrenheit should be provided with an air-chambered vial. Vials designed to maintain a constant bubble length have been tried and found to give inferior results of accuracy. In the instrument used the pattern of the unchambered vial might be adapted with a re-designed bubble reflecting assembly, that is longer 45° reflecting surfaces and consequently thicker prisms that would permit the ends of the shrunken bubble to be reflected onto the reading screen. Such a designed prism box would overcome the present unsuitability of the instrument in extreme low temperature.

"*Minimum Working Temperature.* It has been found impracticable to carry on precise levelling in temperatures lower than 40 degrees below Fahrenheit in a calm dry atmosphere. No opinion can be advanced as to minimum working temperature where there is wind present.

"A mile length of levelling was levelled in both directions at 44 degrees below zero Fahrenheit and the frequent delays that arose from various members of the party having to stimulate circulation and defrost white nose tips, etc., were such that not only was progress painfully slow but accuracy of closing quite unacceptable.

"*Hand Covering.* Three layers of hand covering were used. Woolen knitted gloves covered the hands followed by woolen mittens and finally a suede-finished leather pull-over. The foot screws of the instrument can be manipulated in this ensemble, as well as the directing and clamping of the telescope in position for the rod reading. The manipulation of the micrometer, however, requires sense of touch, and for this operation the gloved hand is removed from the mitten. It has been found that one may operate all day in temperatures not exceeding the above limit. Hand warmers consisting of slow-burning composition in leather-covered metal containers have been tried and discarded as unsatisfactory. Numbness of the fingers has resulted from the use of such devices.

"*Breath Congealing.* The congealing of one's breath upon the eyepiece is perhaps the greatest source of irritation in sub-zero work with a levelling instrument. By adroitly suspending exhalation at the proper moment in much the same way as tobacco chewers time the spitting act in the wind-swept prairie country, much vexation will be prevented.

"The quickest way to remove the opaque white film from the lens is to apply the bare end of the finger to the glass for a second or two.

"*Tripod Setting.* In levelling on ice surfaces a small axe was carried and the ice notched for the iron shoe of each tripod leg. The tripod can be quickly set up in this manner and for temperatures below freezing the bubble exhibits exceptional stability. It follows that under thawing conditions levelling then becomes impossible.

"*Windshield.* A most effective windshield consists of a sheet of light duck six feet long with a channel at either end to receive a pointed iron tube. The person holding the shield stands between two tripod legs facing the wind with outstretched arms, his body taking the pressure on the central part of the shield."

MASTER HIGHWAY PLAN FOR METROPOLITAN BOSTON

By Joseph K. Knoerle*

(Presented at a meeting of the Transportation Section of the Boston Society of Civil Engineers, held on April 28, 1948.)

Foreword

On August 9, 1947, the Honorable Robert F. Bradford, Governor of the Commonwealth of Massachusetts, by Directive, appointed a Joint Board to prepare a Master Plan of Highways for the Boston Metropolitan area. This Board consisting of the Commissioner of Public Works, Chairman of the State Planning Board and Commissioner of the Metropolitan District Commission, on February 1, 1948, presented their report on this subject to the Governor together with a report prepared for the Joint Board by its consulting engineers, Charles A. Maguire and Associates of Boston, and their affiliates, DeLeuw, Cather and Company of Chicago and J. E. Greiner Company of Baltimore.

This report recommended a Master Plan for Highways to include 23 cities and towns in addition to Boston. The consultants were supplied with data compiled from the excellent origin and destination traffic survey which had previously been conducted by the Department of Public Works in cooperation with the Public Roads Administration, Federal Works Agency.

You are all familiar with the complex transportation problem confronting Metropolitan Boston. Many reports and studies of this subject have been made. The Governor and his Joint Board have concluded that now is the time for action and that past accomplishments must be reviewed, information assembled, an integrated Master Highway Plan adopted, funds appropriated and work started at the earliest possible date.

The traffic congestion in greater Boston is no worse than in other metropolitan areas of comparable size. All of these large urban centers have prepared or are in the process of developing master plans involv-

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ing the expenditures of huge sums for traffic and highway improvements. Metropolitan Boston, however, is unique in that it is made up of 24 separate towns and cities, each having its own major business, industrial and residential areas. The efficient movement of traffic within the area has become a problem. Traffic improvements have not kept pace with the growth of these communities.

Many of its streets were laid out in colonial days and were intended for no more voluminous traffic than a few wagons per day or a few riders on horseback. These narrow streets are not adequate for the movement of automotive traffic and their use by the modern motor vehicle can only lead to further congestion and an increasing accident rate.

Improvements to traffic facilities have been curtailed during the past two decades as a result of the depression and World War 2. At the same time, however, population and vehicular traffic have increased. The report presented by the Joint Board and its consultants provides a plan for bringing the area highway system up to date in stages consistent with the expected growth of the community.

A system of expressways has been recommended as the backbone of the entire transportation network. This system, however, must be integrated in a practical manner with an adequate plan of surface streets and other traffic facilities. Complimentary plans now being studied by local officials include those for terminal parking facilities, relocation of certain markets, and extensions and betterments to the rapid transit system. The goal of the metropolitan area should be to improve its entire transportation system in order that the flow of life blood to these communities will not be impeded.

Planning Procedure

The procedure used in studying a problem of this magnitude involves first, an analysis of the facts derived from the origin and destination study which forms the foundation of the entire master plan. Much additional data was made available through separate studies by the planning agencies in the 24 towns and cities throughout the 400 square mile area covered by the report. Considerable time was spent in the field studying various traffic facilities, characteristics of traffic, making reconnaissance surveys of potential locations, studies of many alternate routes, including estimates of cost for construction, right-of-way and traffic evaluation.

Origin and Destination Data

The information collected from the origin and destination surveys makes it possible for the first time to learn where the people go, the time pattern of their travel, trip purposes and modes of transportation. Analysis of this information provides one of the major basis for the selection of routes to best serve the traffic needs. Traffic volumes which would use a proposed expressway and its access facilities can also be determined.

From this data it is possible to establish a desire line of travel which can be defined as a straight line between a point of origin and a point of destination for a trip or group of trips without regard to the actual routes traveled via existing streets. In other words, a desire line is the line of travel which would result were a straight line highway in existence between the point of origin and point of destination. The plotting of these desire lines provides a pattern for the location of expressways from the standpoint of traffic service. These movements are segregated into flow lines between roadside stations outside of the study area and zones within the area; movements between zones within the area and downtown Boston, and movements between zones within the area exclusive of downtown Boston.

Major desire lines of travel for all types of motor vehicles in trip volumes of 100 and over per day have been plotted between the roadside stations outside of the study area and all zones within the area. These desire lines represent 177,000 trips, or 22% of the 811,000 daily trips within the study area. There are 6 or 7 major bands penetrating into the Metropolitan area from important state highways namely: U. S. Route 1 to the northeast; State Route 28 to the north; State Route 2, the Concord Pike to the northwest; State Route 9, the Worcester Turnpike to the west; and Route U. S. 1, and State Routes 138 and 28 to the south. It is surprising to note the small number of movements to downtown zones from the northeast via Lynn and from the southeast via Quincy indicating that many of the trips by individuals from those zones are made via the suburban rail service.

The second grouping of desire lines are those for all types of motor vehicles in volumes of 100 trips per day and over, covering inter-zone trips between all zones in the area and downtown Boston. All inter and intra-zone trips within the area amount to 625,000 trips or 77% of the total of 811,000 daily motor vehicle trips. Of this

total 150,000 trips or 18% are between the downtown area and other zones. The major movement of motor vehicles to downtown Boston is from the west and southwest. This is no doubt due to the large percentage of motor vehicles registered in these areas, the availability of better highways and the inadequacy of rail and mass transportation facilities.

The desire lines resulting from all types of motor vehicles traveling between zones exclusive of downtown amounts to 367,000 trips daily of 45% of all vehicle movements. There is a large local traffic movement in each of the cities and towns surrounding Boston. This is quite apparent in Quincy, Brookline, Watertown, Waltham, Cambridge, Winchester, Malden and Lynn. A close study reveals the semblance of a belt movement of inter-zone traffic surrounding downtown Boston.

The remainder of the traffic within the area consisting of intra-zone movements, amounts to 117,000 trips daily or 15% of all area movements. These movements are not so important as this traffic must always use the local street systems.

To further analyze the downtown Boston problem, a separate study was made to determine the dispersion within this zone of motor vehicles from each of the 7 different surrounding areas. The movement between area 3 and downtown Boston is typical for all other areas; however, in this case the movement is considerably heavier. The center of gravity of all origins and destinations in downtown Boston is in the vicinity of Devonshire and State Streets. This definitely indicates the need for an expressway connection between areas to the west and the downtown business center.

Figure No. 1 illustrates dispersion within the downtown area—the black circles show the origin or destination of all types of motor vehicles from zones to the downtown area, from roadside stations to the area and intra-downtown area movements. A large movement is concentrated near the uptown hotel and shopping district adjacent to Boylston and Arlington Streets. As might be expected concentrations near the North and South stations are heavy. There is also a large focus of traffic near Clinton street, the downtown wholesale market area. Other concentrations center along Washington Street through the shopping center to the financial district bordering Milk Street. It is important to note that very little traffic is evident north of Hanover and Beacon Streets except at North Station.

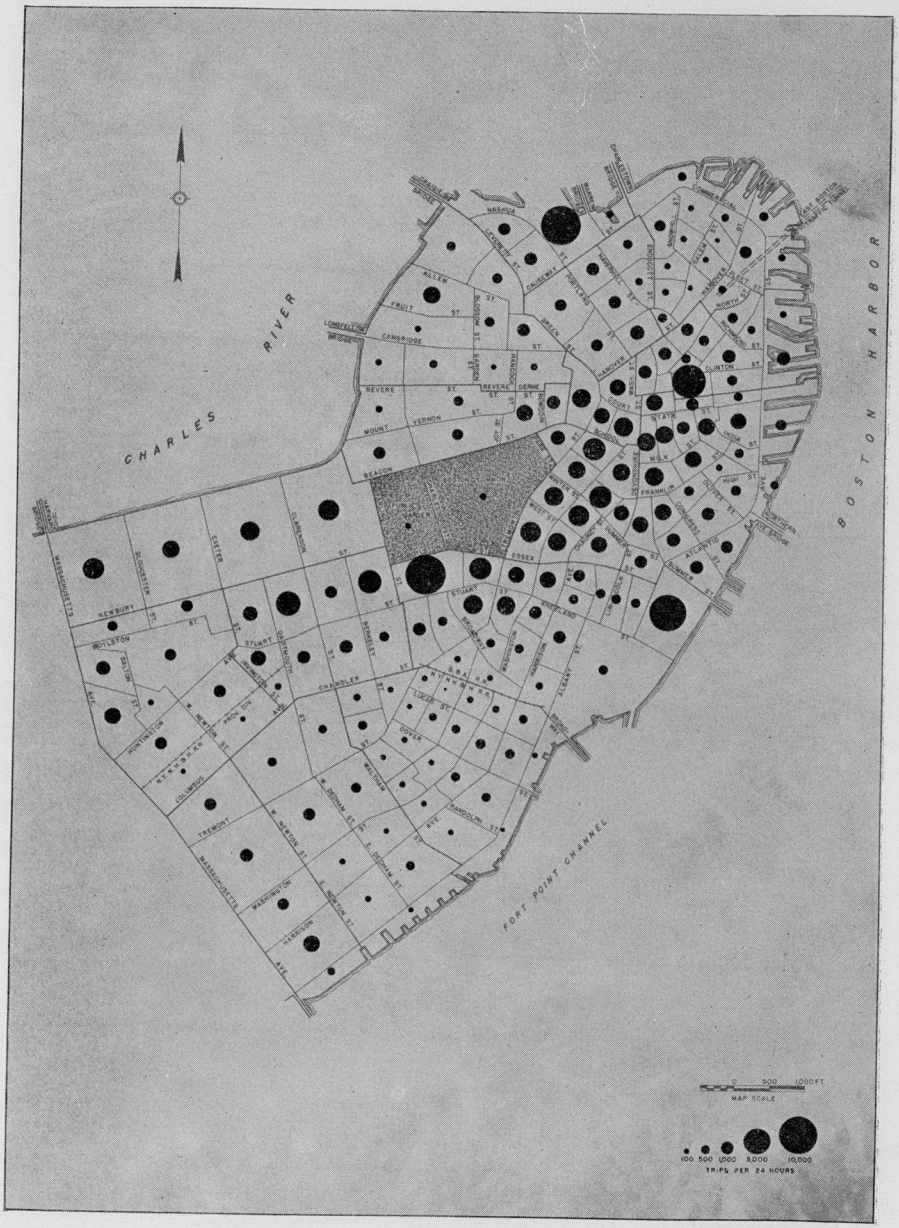

CHARLES RIVER
BOSTON HARBOR
FORT POINT CHANNEL
MAP SCALE
0 500 1000 FT
100 500 1000 5000 10,000
TRIPS PER 24 HOURS

FIG. 1.

From this pattern there can be determined proper locations for expressways, arterial highways for the proper collection and dispersion of traffic, and adequate terminal parking facilities to serve existing and future traffic.

By grouping the various desire lines for all traffic movements a definite pattern of 13 major desire lines is obtained. There are 11 radial movements terminating in the heart of downtown Boston, the heaviest being from the west. There are also large local movements shown for the Lynn, Quincy and Cambridge areas. The remaining two major desire lines indicate heavy crosstown movements from the Quincy area to the northwest crossing the Charles River in the vicinity of the Cottage Farm Bridge, and an east-west movement passing through Cambridge and Charlestown to Revere in the vicinity of the Mystic River Bridge.

While these desire lines are truly representative of the movements of present day traffic, they cannot be considered as the only criterion on which to base expressway locations. Such locations might prove to be unwise should there be major changes in urban developments or land use. In fact, existing traffic patterns might change materially in the future particularly in a complex metropolitan area represented by the 23 towns and cities surrounding Boston.

The Expressway System

In laying out a system of expressways to serve present and future traffic, many factors must be considered in addition to the locations as indicated by the desire lines. It is impractical from a financial standpoint to plan an expressway to serve traffic represented by each of these major desire lines. Several therefore, must be combined in order that one expressway may serve the traffic represented by two or more desire lines.

It is necessary to evaluate existing highways in order that those which are adequate may be used as feeders or supplemental routes, thus minimizing the number of expressways ultimately required to serve future traffic volumes. In general the expressways have been located so as not to parallel or compete with existing facilities. The system was located to provide proper connections with planned state and federal highways. Before exact routes were selected a study was made of many factors which might influence expressway traffic. These included future population increases; zoning ordinances; stability of

existing developments; rehabilitation of blighted areas; trend of industrial expansion; decentralization factors; development of new centers of traffic generation; rail, water, and air transportation plans; possible locations for truck, bus, rail and market terminals; and finally topographic considerations.

Design Features

The expressways planned will require wide rights-of-way varying from 200′ to 300′ in width. Locations therefore have been selected, where such takings of real estate will not entail prohibitive cost. Long sections of sparsely developed property have been found suitable for this purpose. Where populated areas must be traversed, routes have been selected generally in neighborhoods where real estate values are low and are in the process of declining. The new service provided by the expressways should arrest the deterioration of such neighborhoods and help in their rehabilitation.

There are two right-of-way problems confronting public officials in planning and constructing expressways through urban areas which must be properly dealt with if a successful program is to result. First, the right-of-way selected and to be used in a long range plan must be safeguarded and controlled in order that prior to its taking, it will not be developed and its value increased by new construction. To overcome this, some states have proposed passage of a "Highway Development Rights Law". Second, where large numbers of persons must be displaced by the new construction, a redevelopment program is essential in order that these families may be rehoused in suitable locations in an orderly manner. Such a plan has operated in the New York area with great success during the past few years in connection with the construction of several of their major super highways.

Basic design standards utilized in the location and design of the expressway system are generally in conformity with those proposed by federal and state governments for use on the National Interstate Highway System. These standards are necessary because of the large volumes of high speed mixed traffic which will use such highways. They incorporate many features designed to reduce the high accident rates now prevalent on most of the crowded main arterial highways in the Boston Metropolitan area. Expressways must be designed on the limited access principle, wherein specially designed points of egress and ingress are provided at the major traffic

generating centers. For traffic volumes of over 20,000 vehicles daily three traffic lanes in each direction are required. In most of the urban sections service roads for use of the abutting property owners are furnished. Grade separations are provided at all important intersections. The design features shown will tend to eliminate the majority of accidents caused by medial, marginal, intersectional and interstream friction.

Adequate standards for horizontal and vertical curvature and sight distances must be adhered to in order that vehicles may travel such urban highways safely at speeds of 50 miles per hour. Profiles are more or less fixed by existing terrain. Maximum 3% grades are necessary where truck usage of the highways is contemplated.

On certain sections, including the central artery portion through downtown Boston, an elevated highway design is used so that existing streets can remain intact for use of local traffic. Attendant features such as guard-rail, right-of-way fencing, roadway lighting and adequate signing, are provided to facilitate the safe free movement of traffic at all times.

Directional interchanges must be used to connect the various expressways. Typical of these is an interchange merging the southeast and the southwest expressway with the Belt Route in the vicinity of Massachusetts Avenue in Boston.

Another type, commonly called a Delta Interchange, is located in Cambridge to provide for movements between the western expressway and the Belt Route.

Expressway System in Relation to Major Desire Lines of Travel

Based upon the previously described standards and criteria a complete system of multi-lane access expressways was developed. Eight radial routes were selected which coincide generally with the major desire lines of travel. These radial expressways connect to a central belt route which surrounds the downtown section of Boston and part of Cambridge.

To the southeast an expressway parallels the Old Colony Parkway, bypasses Quincy, and connects with Routes 3 and 18 to the South Shore, New Bedford and the Cape Cod area.

To the southwest it was found advisable to provide one route lying halfway between the Hyde Park - Dedham area, with a con-

nection via the Neponset River Parkway to Route 138. The southern terminus of this expressway connects with U. S. Route 1 to Providence.

To the west two expressways are required to handle the large volume of traffic in this area. It was found inadvisable to develop the Worcester Turnpike to limited access standards because of steep grades and right-of-way difficulties. This highway, however, should be improved as an important adjunct to the expressway system. A new parallel expressway was developed to the north following the Charles River to pass through Watertown and Waltham. This new route will supplement Route 9 by providing connections with Route 30 and U. S. Route 20.

Traffic from the northwest now follows the Concord Turnpike and the Alewife Brook Parkway in gaining access to downtown Boston via streets south of the Charles River. This traffic combined with additional traffic from the Winchester-Woburn area is of sufficient volume to require a northeast expressway through Cambridge and Somerville to connect with the Belt Route. This expressway will also connect to a future extension of U. S. Route 3 to Lowell and State Route 38.

Traffic from north to south through the central part of the metropolitan area is generally served by five routes namely; the northern artery, Fellsway west, Fellsway east, Main Street in Malden and Broadway in Everett. The northern artery through Somerville presently carries the heaviest concentration of traffic in the entire metropolitan area. Everett and Malden do not have adequate arterial routes and traffic from these areas must now travel via the Alfred Street Bridge, congested Sullivan Square and Rutherford Avenue, enroute to downtown Boston. It was found necessary, therefore, to develop a new expressway to the north which parallels the desire lines passing through the Malden, Everett and Melrose areas. This route is extended to bypass Stoneham and Reading and connect with Route 28, a heavily traveled state route to the north. The southern end connects to and utilizes the existing northern artery between downtown Boston and the Mystic River. It was determined that this latter section could be developed to expressway standards more economically than should an entire new location be utilized. Because this northern expressway is expected to carry one of the largest volumes of traffic in the network, it is connected to the central area of Boston independently rather than via the inner belt and central

artery. In this manner, traffic distribution in downtown Boston will be facilitated.

Two major desire lines extend to the northeast through the municipalities of Everett, Chelsea, Winthrop, Saugus and Lynn, connecting with North Shore points and U. S. Route 1, the Newburyport Turnpike. A location for one expressway lying between these two lines was decided upon which coincides exactly with the location of the new Mystic River Bridge now under construction. By connecting this bridge project to U. S. Route 1, State Route 107 to Lynn and Route 1-A to Revere Beach, a complete facility is provided for travel to downtown Boston and other points in the metropolitan area via the Belt Route.

Two major desire lines terminate in the East Boston, Revere area. The traffic volumes indicated are lighter than on other expressway sections. The movements are also relatively short between termini. Presently, this traffic is served by the two-lane Sumner Tunnel which connects with Logan Airport and Route 1-A to the northeast. This facility is now overloaded by traffic detoured from other less desirable routes. The proposed East Boston elevated highway and expansion of the Logan Airport within the next few years should increase the traffic demands on the Sumner Tunnel. Upon completion of the Mystic River Bridge, a considerable amount of this traffic will be diverted to that facility. It is felt, however, that within the next few years, volumes will have increased to a point where an additional traffic facility across the harbor will be required. This traffic can be handled adequately by an additional two-lane tunnel paralleling the Sumner Tube. In this manner the two tunnels can operate as a pair of one-way arteries between downtown Boston and East Boston.

In order to provide a bypass for traffic around the downtown area and to facilitate the distribution of traffic to the major streets from the radial expressways, a belt expressway is mandatory. The radial routes are fairly well distributed around this entire belt. Property values, growth of the central area, access to important downtown points of origin and destination and connections to important traffic arteries were prime considerations in locating the belt expressway. This route, however, must be supplemented with an adequate system of improved and widened one and two-way surface thoroughfares if it is to function properly as a distributing artery.

Beginning at a point near City Square in Charlestown the Belt Route connects to the Mystic River Bridge Project, passes south over the Charles River, and thence via an elevated highway connects with the Sumner Tunnel, passes by the South Station, parallels Albany Street and joins with the southeast and southwest expressways in the vicinity of Massachusetts Avenue and Southhampton Street. From that point it extends to the west through Roxbury to connect with the Worcester Turnpike at Brookline Avenue, thence to the northwest across the Charles River just west of the Cottage Farm Bridge to connect with the western expressway in Cambridge. Thereafter it extends north through Cambridge to connect to the northwest expressway, turns east to cross the northern artery, passes over the Boston and Maine Railroad yards, and joins the northeast expressway at the Mystic River Bridge.

An adequate number of ramps are provided at intervals along this facility so located that in no case will surface streets be congested by the discharge from these ramps. The belt route is somewhat larger than has been found necessary in other cities where comparable studies have been made. However, it must be considered than an unusual number of radial routes are involved, that extensive areas of water are encompassed and that several distinct business centers rather than the usual one are served. Traffic volumes on this belt are very high. A six-lane facility is required to handle movements which are expected to vary from 40,000 to 80,000 vehicles daily by 1970. In order to handle this volume an extra lane is considered necessary in some instances to serve the acceleration and deceleration movement.

The traffic analysis shows a heavy desire line paralleling the Charles River Basin. Much of this traffic is predominantly local in character and now uses Commonwealth Avenue and Beacon Street. These streets are badly congested due to cross conflicts. To improve this situation the Metropolitan District Commission has proposed the construction of a new facility, a six-lane divided highway of modified limited access design for automobile traffic known as Embankment Road. It will extend along the Charles River from Bay State Road to Charles Street. This project in supplementing the Belt Route will serve a useful function in moving traffic between downtown Boston and other areas within the Belt. It has therefore been included as an essential part of the expressway system.

The complete expressway system comprising the 8 radials and the Belt Route is shown in relation to the Boston area is shown in Figure No. 2. It can be easily seen how well the entire area is served for the major flow of heavy trunk movements in all directions. This will be particularly appreciated by truck traffic, movement of which is seriously impeded in this area because of the lack of direct through routes of adequate capacity.

Downtown Boston

Figure No. 3 shows the central artery section of the belt expressway previously described together with recommended improvements for certain arterial connections and a one-way street system for downtown Boston. Various alignments for the central artery have been previously proposed and a number of alternate locations were surveyed by the consultants before arriving at the location shown. The route selected differs somewhat from those previously proposed primarily because of the need for proper connections to radiating expressways, completion of the Belt Route, and connections via ramps to existing arterial streets including the Sumner Tunnel. The route of the central artery which starts in downtown Boston just above the present Warren Bridge is in a southerly direction crossing Haymarket Square to run parallel to Atlantic Avenue. The most difficult problem in connection with the central artery is the location of ramp facilities. Distribution must be from both sides of the facility into the downtown area where traffic desires to travel. A one-sided location along the waterfront cannot accomplish this purpose.

A number of street widening projects are shown together with an adequate plan for one-way movements. Supplementing these improvements, there is need for a system of north-south cross town one-way arteries to connect the Embankment Road project with the best expressway. Arlington, Berkeley, Clarendon and Dartmouth Streets are admirably suited for such purposes. These facilities, when supplemented with an adequate system of parking terminals should relieve the present traffic congestion in the downtown area to a great extent. Alone, however, these cannot be expected to accomplish this objective unless a well conceived vigorous program of enforced parking regulations is instituted on many of the arterial and one-way streets shown. Such a program is needed not only in downtown Boston, but in essentially all of the surrounding cities and towns.

FIG. 2.—METROPOLITAN MASTER HIGHWAY PLAN.

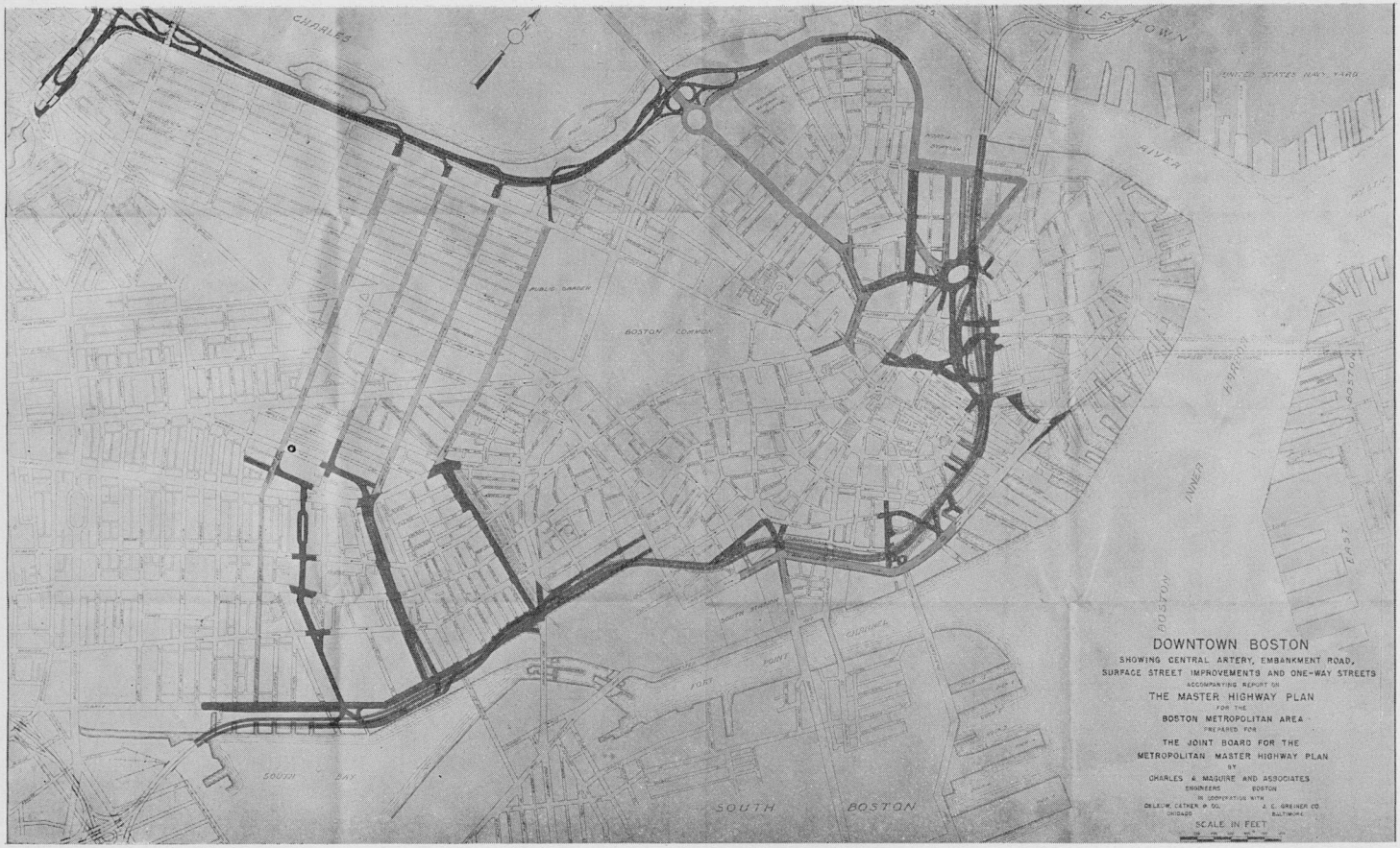


FIG. 3.—CENTRAL ARTERY, EMBANKMENT ROAD, SURFACE STREET IMPROVEMENTS AND ONE-WAY STREETS.

Fines and enforcements must be severe enough to deter repeaters and flagrant violators, and special parking privileges nullifying restrictions should not be tolerated. If this is not done, traffic congestion will cause further decline in property values which will amount to many millions of dollars in excess of the cost of the proposed improvements. If the area is not made accessible, further decentralization must be expected. We all know that in Boston, like in many other cities part of the congestion is caused by illegal parking and when viewed in this light, the seriousness of the acts of parking violators is clearly apparent.

Estimates of Future Traffic

The origin and destination survey indicates volumes which would use an expressway system on the basis of 1945 traffic figures. Projections of future traffic must be carried to at least the year 1970 to determine the trend in different portions of the area to preclude the possibility that the highways will be either over-designed or under-designed. Population changes and increases represent one of the most important factors in arriving at traffic volumes. Each local area was studied to determine the influence certain changes might have on the population in that area. Trends in vehicle registration and motor car usage have also been taken into account in arriving at the 1970 volumes. From the above factors a basic increase of 1.77% over 1945 traffic was determined. This factor has been tempered in each case by the effect of population predictions in each area wherein different route factors have been figured. These volumes vary from 33,400 to 64,150 vehicles daily in 1970 at the belt connection.

On the Belt Route the volumes are much heavier varying from 41,000 to 81,000 daily in 1970. This traffic being so heavy it emphasizes how important will be the function of the local street system to absorb and disperse this predicted traffic. It should also be remembered that much of the traffic which now uses the existing narrow streets will be removed upon completion of the central artery and it certainly does not take much imagination to visualize the resulting relief.

Estimates of Cost

Complete project costs for each mile of expressway was determined from preliminary plan and profile maps. The total length of

the 8 radial expressways and the Belt Route is 85.4 miles. Total project costs based on current prices amounts to $278,400,000. These costs are made up of items for preliminary work; construction costs and contingencies; engineering; demolition and moving of buildings; changes and relocation of utilities; legal, administrative and overhead costs; maintenance of traffic; and assessed valuations of right-of-way, together with engineering, legal and administrative costs, in connection with property acquisition. Preliminary costs include those items generally incurred on large projects prior to the preparation of contract plans and specifications, and cover such items as subsurface explorations and additional traffic studies.

Construction costs are based upon the design standards heretofore described. Costs are based upon an analysis of conditions at each site, study of available local materials, current bid prices and contractors and suppliers costs on work of similar character. To arrive at the basis for estimating quantities, it was necessary to prepare strip plan and profile maps for each mile of expressway. These plans were prepared from available city, town, topographic and real estate maps for every area traversed with due consideration to existing and proposed utilities. Locations could not be decided upon until alternate studies were made to compare right-of-way values in each instance.

Right-of-way costs form a large part of the total project cost, and of course vary according to the character of the properties traversed. In estimating these costs field surveys were made and assessment records secured in each of the towns and cities affected. The estimates of right-of-way are based on currently assessed valuations for both land and buildings. Costs incidental to the acquisition of right-of-way cover surveys, preparation of property plats, appraisals, searching of records, court costs, moving of tenants, and other expenses, which normally amount to 15% of the assessed valuation. Right-of-way costs for the entire expressway system amount to $67,500,000 or 24% of the total project cost. Construction costs, which include contingencies, amount to $172,650,000 or 62%. The remaining miscellaneous items of cost including preliminary costs, changes in utilities, engineering, demolition and maintenance of traffic amount of $38,250,000 or 14% of the total project costs.

Construction Procedure

The scheduling of a program of the magnitude described requires the coordination of legal, financial planning, engineering, contracting, traffic demands, and other controlling factors in order that the work may be prosecuted smoothly in stages consistent with the Commonwealth's desire and ability to finance such a program. The traffic analysis indicates volumes of sufficient magnitude to require the construction of a major portion of this program immediately. It was not within province of the consultants to determine the amount of funds which could be made available from year to year and therefore a study was made to set up a schedule for constructing usable sections according to the relative urgency of the needs of the various areas.

It appears that a maximum of 5 stages of work should be planned. Each stage includes construction of portions of 3 or more expressway projects. Thus work would be in progress simultaneously on widely separated sections thereby minimizing interference with existing traffic and other services. In setting up such a priority program the first unit of an individual expressway must serve as an efficient traffic artery alone until the entire expressway system is completed. It is also important to make sure that the construction of a single section of expressway may not impose conditions upon the existing street system tending to cause further congestion. The first stage includes the central artery, connecting to the Mystic River Bridge and the Old Colony Parkway, the East Boston Elevated Highway and Embankment Road.

The second stage includes extension of the northeast expressway to U. S. 1 and the North Shore routes, extension of the central artery and Belt Route via the northwest expressway to the Concord Turnpike, improvements to existing Route 9, the Worcester Turnpike and connection with the Belt Route via the southwest expressway to Blue Hill Avenue and Washington Street.

The third stage includes the completion of the Belt Route and second tube of the Sumner Tunnel and extension of the southwest expressway to its southern terminus at U. S. Route 1.

The fourth stage includes the northern portion of the northern expressway to connect the Stoneham Bypass to the existing Fellsway, reconstruction of the northern artery from the Charles River to the

Revere Beach Parkway, extension of the western expressway from the Belt Route to Galen Street in Watertown and construction of the southeast expressway from the Old Colony Parkway and Gallivan Boulevard to its southern terminus connecting with Routes 3 and 18.

The fifth and last stage includes widening and improving of the Newburyport Turnpike as a limited access highway as far as its intersection with Route 128, completion of the northern expressway from the Revere Beach Parkway to Stoneham, completion of the outer portion of the northwest expressway from Massachusetts Avenue through Winchester and Woburn to connect with Routes 3 and 38, completion of the western expressway from Galen Street to connect with Route 30 at Commonwealth Avenue and completion of the middle portion of the southeast expressway to replace that section of the Old Colony Parkway previously used for this expressway during the first four stages of construction.

It would be possible to perform all of the planning and construction work for this entire system within a ten year period. The average annual funds needed for such a program would be approximately $28,000,000. The entire expressway system could be completed by 1958 and in the meanwhile motorists of the area would be enjoying the benefits derived from their use as the work progresses.

Economic Justification

The consulting engineers were not required to outline a formal financing program for the expressway system. They did, however, make a study of the economic benefits to the users which can be weighed against costs for constructing such facilities. Expressways have proved to be a good investment for the motorists and taxpayers in other metropolitan areas and there is no reason to believe such should not be the case in the Boston area. The use of such highways will result in great savings in time for the many millions of vehicles using them annually, while at the same time surface streets will be freed of unnecessary congestion to speed up the movement of vehicles and people. As an example, it is estimated there will be 75 million vehicle miles of travel on the central artery by the year 1970. Based on existing conditions the time saving in traveling through this area will be at least four minutes per mile. The value of a motorist and trucker's time can be set conservatively at 1¼ cents per minute, thus the annual saving on the central artery alone by 1970 would amount

to $3,750,000. This benefit capitalized 3% over a period of thirty years would justify an expenditure of approximately $74,000,000 which exceeds the cost of this section of the expressway. While direct dollar benefits can prove that expressways are justified, there are many others of even greater significance which cannot be so readily estimated. These consist of social benefits including the assurance that traffic accidents will drop to a fraction of the rate current on surface streets. If these highways are not built, many expensive temporary expedients must be instituted which experience indicates does not satisfy the need for modern traffic arteries. Finally, it is believed the greatest economic value the expressway will provide would be the protection and enhancement of property values throughout the area. The prosperity and welfare of every community depends on the expeditious movement of people and goods.

The foregoing will give some idea of the various steps necessary in formulating an overall expressway plan for a large metropolitan area. The subsequent stages of financing and construction should be prosecuted in a vigorous manner by public officials charged with this responsibility. The path ahead is not an easy one.

If the urban plan of living and doing business as we know it is to operate efficiently, such steps to improve the area transportation system should be instituted immediately.

OF GENERAL INTEREST

C. FRANK ALLEN
1851 - 1948

By Lewis E. Moore

C. Frank Allen, whose life-span covered almost a full century, has passed from the sight of his friends. His lifetime included the period in which mankind has made greater material progress than in all of the preceding known portion of human existence. He was a part of this progress, always keenly interested in the new, but never making the mistake of losing his grasp on the ageless fundamental human and spiritual values.

He was born July 10, 1851 and passed on June 6, 1948.

His full name was Calvin Francis Allen and, while he was generally known as C. Frank, he used the name Calvin F. Allen for formal matters such as deeds of land, voting lists, etc. His recorded ancestry goes back seven generations to Lewis Allen who settled in Weston. The family moved to Dedham and shortly thereafter to Walpole which he always regarded as the seat of the family. His mother was from a family named Watson which is recorded in Paige's History of Cambridge. Prof. Allen followed Dr. Holmes' receipt for longevity, which was to select a long-lived series of ancestors. All in a direct line varied in ages at their deaths from past eighty to ninety-nine and three-quarters.

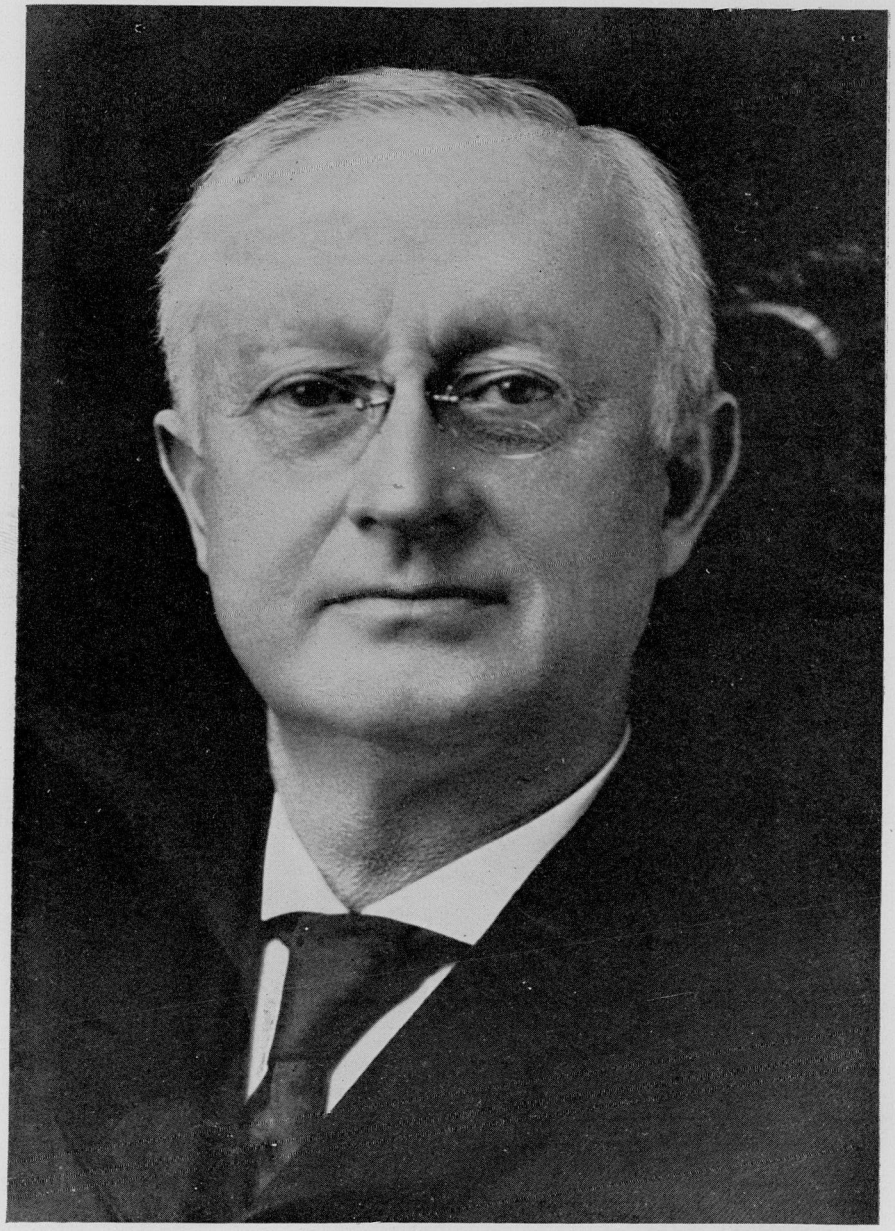
He received his high school education at the Roxbury Latin School and then attended the Massachusetts Institute of Technology from which he was graduated in 1872. At the M.I.T. Alumni Dinner in 1947 he represented his class which had graduated 75 years previously.

During his summer vacations while at M.I.T. he did work in surveying in the Boston office of J. Herbert Shedd. His first engineering work after graduation was as a surveyor and later as assistant engineer with the Providence Water and Sewerage Works after which he was appointed engineer in charge of the construction of the Newton Reservoir. The following year he was put in charge of the extension of the filter basin at Newton Upper Falls and carried that work to a successful completion.

The severe depression of the seventies altered the direction of his career and he found employment, lasting for some six years, with the Atchison, Topeka and Santa Fe Railroad. His employment in the West also included a year as Chief Engineer of the Las Vegas, New Mexico, water works and some time with the Mexican Central Railroad at El Paso.

The character of his work with the railroad brought him into close contact with certain legal questions which led him to study law because of his inherent desire to know the fundamental principles underlying whatever work he undertook. As a result he became a member of the New Mexico Bar and later of the Massachusetts Bar. His two years of practising law included serving as local attorney for the Atchison, Topeka and Santa Fe Railroad and as city attorney of Socorro, New Mexico.

He returned to the East in 1877 and



C. FRANK ALLEN
1851-1948

at that time accepted the invitation to take charge of the instruction in Railroad Engineering at M.I.T. He retained this position for 29 years until his retirement in 1916.

He was the author of the following books which are recognized as the authoritative standards in their lines: *Railroad Curves and Earthwork; Field and Office Tables;* and *Business Law for Engineers.*

He was a member of various associations and was actively engaged in committee work in many of them. They comprised the following:

American Society of Civil Engineers, of which he was the second in seniority and an honorary member.

Boston Society of Civil Engineers, of which he was, in addition to being an honorary member, the senior member and past president.

New England Railroad Club, of which he was a past president and an honorary member.

Massachusetts Highway Association of which he was a charter member, senior past president and an honorary member.

American Society for Engineering Education, formerly the Society for the Promotion of Engineering Education, of which he was a past secretary, senior past president, one of the founders, and one of a committee of three to write the constitution and determine the name.

American Railway Engineering Association, of which he was a life member.

M.I.T. Alumni Association of which he was at one time the secretary.

Technology Alumni Council.

Secretary of class of 1872 M.I.T.

He was a member of the Boston City Club, the Highland Club of West Roxbury, the Faculty Club of Technology, and was secretary of the old Technology Club.

He was a 32d degree Mason.

That his interests were diverse is instanced by being chairman of the school committee at one time while he was living in Sharon and by playing a good hand at contract bridge every Saturday night at the Highland Club until January 1948.

He received the honorary degree of Doctor of Engineering from Northeastern University in 1938.

On the seventieth anniversary of his graduation from the Roxbury Latin School, in 1938, he gave the Commencement Address at that institution.

In 1888 he married Caroline E. Hadley who survived him by about three months. Surviving are two daughters, Mildred Allen of Mount Holyoke College and Margaret Allen of West Roxbury.

Mrs. Allen, a birthright friend, was born in Richmond, Indiana, and met Professor Allen in New Mexico where her father was President of the New Mexico College of Agriculture and Mechanic Arts near Las Cruces.

Although he spent nearly thirty years of his life as a teacher, and was generally called "Professor Allen", he was much more than simply a teacher. He once told me that he had made a living in each of the following vocations: engineer, lawyer, author, teacher, and in the stock market. This furnishes an index of his breadth and an indication that he used his active analytical mind to a far greater extent than do the majority of humankind.

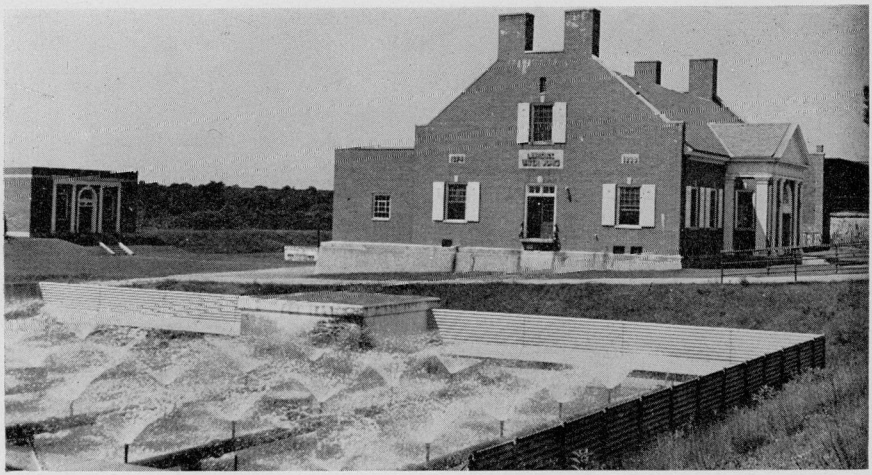
It is difficult to appraise the work of a teacher because the full fruition of his efforts becomes visible only after many years of growth and further development on the part of his pupils. His work consists of the implanting of seeds and the bending of the twig that finally result in the inclination of the tree. The teacher can only hope, when some of his former pupils succeed, that his influence may have had some part in aiding the achievement, and that he can say, to himself only, "I am

proud of my small part in that." I am sure that Prof. Allen spent many a quietly satisfactory moment contemplating the numerous successful men who had been under his tutelage.

So long as his friends endure, he will be held in respectful remembrance for his kindness, keen sense of justice and fair play, his sound but never obtruded advice, and his abiding commonsense.

He possessed, to an unusual degree, studiousness without pedantry, flexibility without indecision, and tolerance without indulgence.

All who came in contact with him were the better for it.



LAWRENCE WATER WORKS

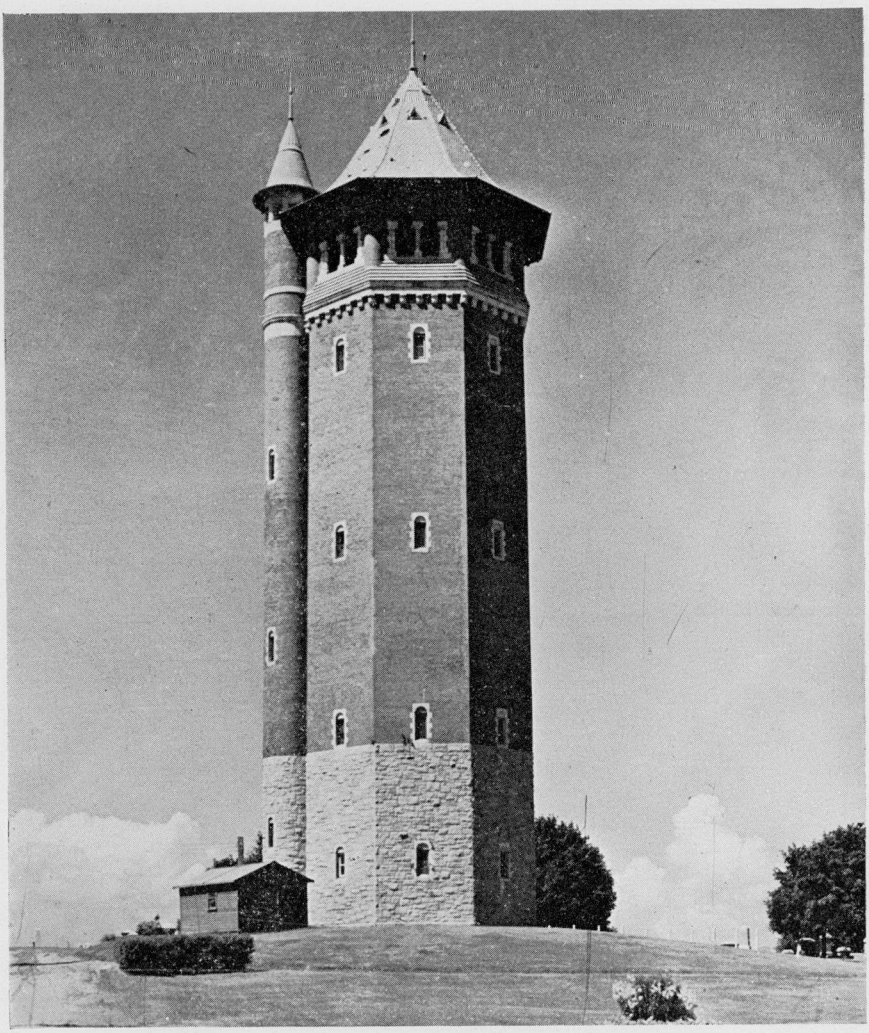
Sanitary Section Inspection Trip To Lawrence, Mass.

On June 5, 1948 the Sanitary Section held their June meeting in the form of an inspection trip to Lawrence where the works of the Lawrence Water Department and the Lawrence Experiment Station of the Mass. Department of Public Health were visited.

Mr. John W. McCarthy, Jr., Water Commissioner of the City of Lawrence, guided the group through the pumping stations and filtration plant, and to the service reservoirs of the City.

Lawrence was one of the first municipalities to adopt filtration of its public water supply in this or other countries, the original slow sand filters being built in 1893 and serving, with additions and modifications, until about a year after the new filtration plant was completed in 1937. Even today the source of supply is still the highly polluted Merrimac River. However, the standard rapid sand filtration plant, now operating at almost exactly its eight million gallon per day designed capacity continuously provides the city with safe water.

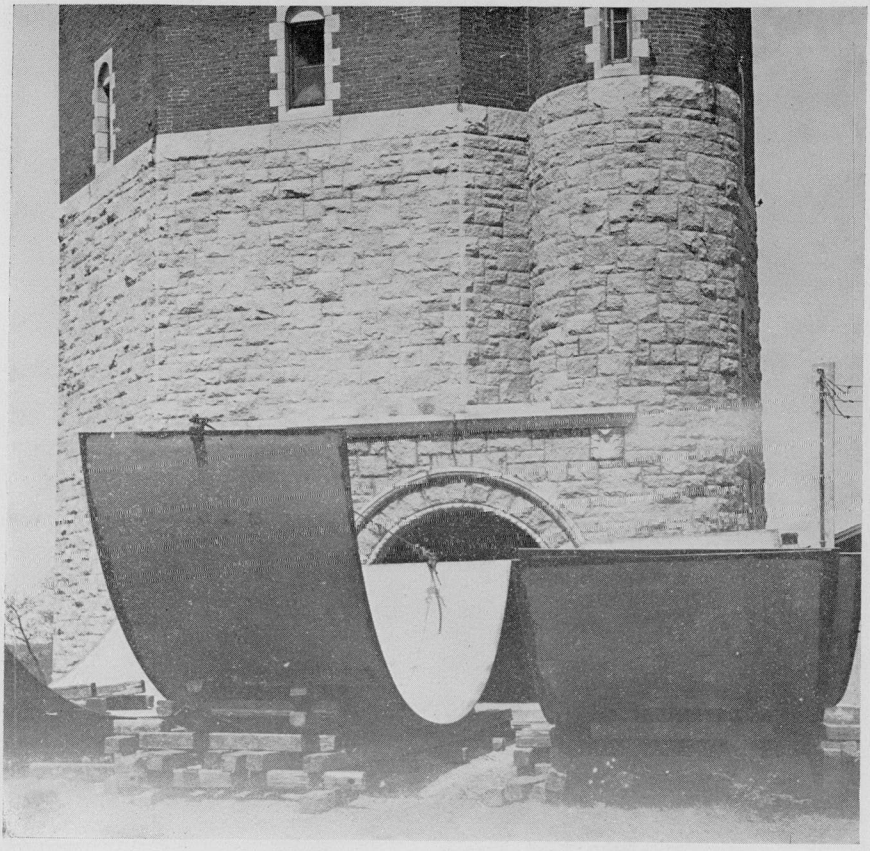
Most notable in connection with the treatment is the discontinuance of the old slow sand filters as secondary filters to the new plant as was provided in the original design. Complete reliance is now placed on the rapid sand plant plus chlorination in a total



TOWER RESERVOIR

amount of approximately three or more parts per million as pre- and post-chlorination.

The visit to the high service reservoir or standpipe proved most interesting, in view of the fact that it was under repair at the time. This reservoir is a granite and brick tower enclosing a 500,000 gallon capacity steel tank. The old tank which was built in 1896 had ended its useful life and was removed. At the date of the trip the new tank was about half completed. All steel was brought in through a relatively small doorway at the base of the tower, hoisted into place, and welded.



CONSTRUCTION VIEW OF 500,000 GAL. STEEL STANDPIPE

Before this tank could be taken out of service it was necessary to install a gasoline engine on one of the high service pumps to care for power failure, since direct pumping was necessary. Except for this one marine type engine the entire plant relies solely on electric power and adequate high level storage.

Mr. Joseph A. McCarthy, Chief of Laboratory, Lawrence Experiment Station spent the afternoon discussing current research investigations under way at the Station.

Extensive experiments are being conducted with ultra high rate trickling filters in an attempt to reach the point at which no purification takes place in the unit. There is also an active program on industrial waste treatment, spurred on by recent legislative action which brings stream pollution under the jurisdiction of the Mass. Dept. of Public Health. Various biological methods of treatment are being tested, along with chemical methods. A striking demonstration of the treatment of a wool scouring waste by carbonation followed by calcium chloride treatment was given.

Mr. William J. McGee, Senior Chemist, showed the laboratory, and described the methods used, where approximately 20,000 bacteriological samples from public water supplies in the state are tested each year.

KENNETH F. KNOWLTON,
Vice-chairman

Data On Flow Tests On Mokelumne Aqueduct

CONTRIBUTED BY EDWIN B. COBB

In the January 9th issue of Engineering News Record there appeared an article regarding flow tests on an aqueduct in California. The writer has been compiling such data over a period of years and seeking a source for some valuable additional information he wrote Mr. H. A. Knudsen, Engineer in Charge of the Mechanical and Electrical Division of the East Bay Municipal Utility District, Oakland, California. Mr. Knudsen's reply was so complete and because such data is quite rare the writer requested and obtained permission to publish it in full in this JOURNAL. The letter is as follows:

On January 16, 1947 you wrote for additional data in connection with an article in the January 9th issue of Engineering News Record. Although by the use of the Kutter Formula we were able to compute values that were surprisingly accurate in this particular instance we were fully aware that this formula would not be adequate in all cases. We had never analyzed the data on the Aqueduct by means of the Moody Formula and since receiving your letter have devoted considerable time to such an analysis.

During the period 1929-1946 five separate tests were made on the flow through the Aqueduct. The pipe is coated internally with an asphaltic dip and is in excellent condition. We have never detected any decrease in carrying capacity. The first test was conducted in 1929 just after the pipe had been placed in operation and was directed by Mr. Fred C. Scobey, of the U. S. Dept. of Agriculture. Similar tests were conducted in 1935 and 1936 by our own organization. The 1929 and 1935 tests were made at very nearly the same flow rates as the one in 1936. The data from these first two tests were not included in this analysis, the results of the 1936 tests being considered superior to the two former tests.

The test flow in 1936 was 42.46 M.G.D. as determined by color velocity observations. Elevations of the gradients were observed directly in the Outlet Tower in the Pardee Reservoir and in the Aeration Weir at the inlet of the Walnut Creek Tunnel. Pressures were determined at intermediate points by means of mercury manometers.

On December 23, 1943 when studies were being made to determine the characteristics for the Bixler pumps, pressures were observed at four points along the Aqueduct between the West Portal of the Pardee Tunnel and the Walnut Creek Pumping Plant. During these observations the flow was measured by venturi tubes equipped with mercury manometers. The calibration for these venturi throats was determined from the previous tests. The flow at West Portal was 66.26 M.G.D. and the Walnut Creeek Pumping Plant 63.59 M.G.D. The difference of 2.67 M.G.D. was due to consumption at Camp Stoneman, near Pittsburgh. Pressures were determined by means of calibrated test gauges.

The most recent data was derived from two successive color velocity tests of June 25 and 26, 1946. The flow was measured by the color velocity measurements in the manner described in a paper appearing in the February issue of the A.W.W.A. Journal. Observations were also made on the measurement of flow by the Venturi meters at West Portal and Walnut Creek. The flow measured by color velocity method was

98.35 M.G.D. at Bear Creek on June 26 and 97.6 M.G.D. at Bixler on June 27. The flow as measured by the venturi meter at West Portal remained practically constant during a period of several days including the time of the tests. The difference between the two observations, namely 1.29 M.G.D. was due to consumption by the Naval Base on Rough and Ready Island, west of Stockton.

Analysis of these measurements is further complicated in that the Aqueduct is composed of 63", 64" and 65" I.D. pipe and approximately 3,000 feet of twin 54" pipe in Three River Crossing. To simplify the analysis we have reduced all of the pipe sizes to equivalent lengths of 65", there being more of this size in the Aqueduct than any of the others.

Water temperatures were not observed on any of these tests. Our Sanitary Department's records show the temperature at West Portal on December 22, 1943 at 55° F. It is estimated at that time of the year the average temperature along the Aqueduct would vary only one to two degrees higher. On June 26, 1936 it is estimated that the temperature was approximately 65° and it is reasonable to assume the temperature was about the same in June, 1946.

Table No. 1 shows basic data and the results of an analysis of these data by means of Kutter's Formula.

We have also, as indicated earlier in this letter, attempted to analyze the data by the Moody and Darcy Formulas. Using the data obtained in 1936 and 1942 the following values were computed:

Flow M.G.D.	V	f	R_e	$\frac{K}{D}$
44.46	2.870	0.01525	1.26×10^6	2.735×10^{-4}
66.26	4.475	0.01506	1.965×10^6	2.775×10^{-4}
Average				2.755×10^{-4}

The following table is based on the above values:

No.	f	VF	R_e	f.p.s. V at 60° F.
1.	0.0148	0.1217	6.430×10^6	14.65
2.	0.0149	0.1221	3.315×10^6	7.55
3.	0.0150	0.1225	2.325×10^6	5.30
4.	0.01505	0.1237	2.005×10^6	4.67
5.	0.01510	0.1235	1.825×10^6	4.16
6.	0.01515	0.1233	1.644×10^6	3.74
7.	0.01520	0.1231	1.463×10^6	3.33
8.	0.01525	0.1228	1.310×10^6	2.98
9.	0.01530	0.1227	1.173×10^6	2.67

TABLE NO. 1

VALUES OF "n" IN KUTTER'S FORMULA AS COMPUTED FROM DATA FROM VARIOUS FLOW TESTS ON THE MOKELUMNE AQUEDUCT

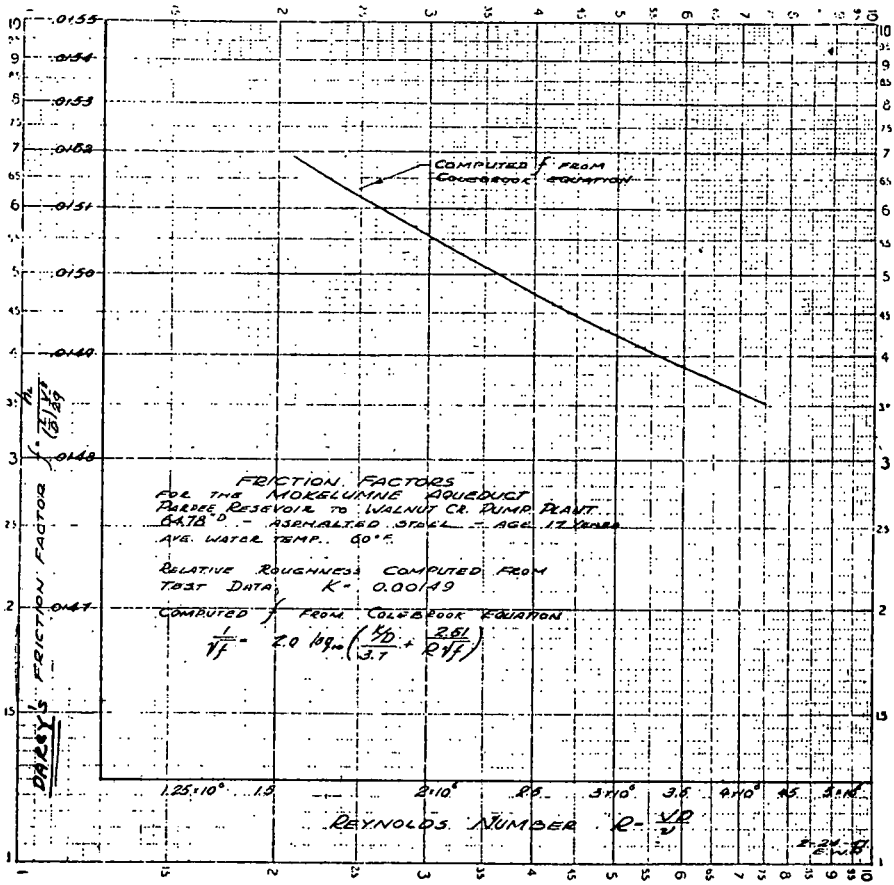
Date of Test	Section of Pipe	Rate of Flow M.G.D.	Equivalent Length of 65" Pipe*	Loss of Head per 1000 feet	Kutter's "n"	Section No.†
June 24, 1936	West Portal to Aeration Weir	42.86	469100	.35737	.01216	All
	West Portal to Bixler at 2711+78	42.86	288750	.36668	.01231	18c 2
	Bixler at 2711+78 to Aeration Weir	42.86	180350	.33585	.01181	38c 4
Dec. 22, 1943	West Portal to Marsh Creek	66.26	310902	.88549	.01236	18c 2
	Marsh Creek to Stoneman	66.26	63055	.80674	.01183	3
	Stoneman to Walnut Ck. P.P.	63.59	88060	.74303	.01183	4
	Weighted average West Portal to W. Ck. P.P. (Weighted by Equivalent Lengths)	——	——	——	.01218	All
June 26-27, 1943	West Portal Navy Base	98.35	195240	1.9020	.01225	1
	Navy Base to Bixler P.P.	97.06	95434	1.8563	.01225	2

*Based on Kutter's Formula. Includes allowance for other losses in the various sections of the Aqueduct. Expressed in terms of equivalent 65 inch pipe these are as follows:

Johnson Valve at West Portal	1750 feet
Each venturi meter, West Portal or Walnut Creek P.P.	1000 feet
Each Wasteway	1600 feet
Each River Crossing estimated entrance and exit losses, in addition to pipe itself	500 feet

†The various sections have been numbered for easier comparison. Sections of same number are only approximate in some instances.

The following graph shows the relation of "f" and "R_e".



THE MOKELUMNE AQUEDUCT

Book Review Section

PRINCIPLES OF INDUSTRIAL PROCESS CONTROL. *By Donald P. Eckman.* John Wiley & Sons, Publishers. 5⅝×8⅝. 237+x pp. numerous figures. $3.50. Published 1945.

REVIEW BY ALLEN J. BURDOIN

This book is a good elementary text on the science of automatic control as it relates to the process and allied industries. Intended primarily for the student of chemical, metallurgical, mechanical, or electrical engineering, it will prove of value also to practicing civil and sanitary engineers interested in the control of pressure, temperature, fluid-flow, liquid level, humidity, and pH.

The book begins with a consideration of various measuring means and their characteristics, including lag, dynamic error, and dead zone, for one must be able to measure before one can control. Then follows a chapter describing the action of the various types or modes of automatic control such as two position, floating, proportional, proportional-reset, and proportional-reset-rate. The action of the control valve and its effect on the controlled variable is emphasized. After this comes a chapter on final control elements in which the characteristics of valves and dampers are discussed from the standpoint of control.

A chapter on process characteristics defines and evaluates those common characteristics of different processes which are important from the standpoint of control such as capacity, reaction rate, transfer lag, dead time, and self-regulation. This chapter is followed by a chapter in which the application of the various modes of control to processes with different characteristics is discussed, and a chapter on the quality of automatic control obtained in response to various types of load changes, and as affected by different controller adjustments.

The rest of the book discusses several common control problems such as flow and pressure control, and automatic control systems in which more variables than one are controlled. These include metered control, ratio control, time-variable control, limit control, and problems involving dual control agents. The book ends with a chapter on maintenance, which includes installation suggestions.

The book includes a glossary of automatic control terms, and over one hundred references to the literature of automatic control.

The mathematics will be simple for anyone with a knowledge of second order differential equations, but those interested persons who are without such knowledge should nevertheless be able to derive great benefit from the book.

The book is concerned throughout with fundamentals. No descriptions of actual controllers or catalog information is included, since this is readily available from the various instrument companies. Such descriptions as are included are representative of a type, and only sufficiently detailed to illustrate the fundamental ideas involved.

This reviewer believes that the fundamental reason for measuring and recording process variables is to be able to control process or plant operation satisfactorily. He further believes that this can usually be done better by automatic than by manual means. Automatic control to be successful, however, must be properly engineered and this engineering should begin with the project engineer, who must know how to control his process, as well as what types of equipment industry has to offer, in order to cooperate successfully with instrument experts. This book can provide him with the necessary "know-how."

John R. Freeman Fund Scholarship Awards

The John R. Freeman Fund Committee has awarded two scholarships this year, the first starting October 1, 1948 and the second July 1, 1949. These are as follows:

Ralph S. Archibald is a resident of Melrose. He was born in Malden in 1924. After graduating from high school in Dover, New Hampshire, in 1942 he entered Tufts College School of Engineering in the Naval Reserve Officers Training Corps. He graduated from Tufts with the degree of B.S. in Civil Engineering, Summa Cum Laude, in 1945. He served in the Navy until July, 1946, and entered the Harvard Graduate School of Engineering where he obtained the degree of Master of Science in Sanitary Engineering. He was the winner of the Clemens Herschel Prize in Hydraulics for that year. He is a junior member of the Boston and American Societies of Civil Engineers.

Mr. Archibald's project is research in "Radioactive Tracers in Flow Tests". The tests will be based on the possible uses of certain radioactive substances introduced into fluids, and their movements traced by the use of Geiger tube counters which are able to detect the radiations given off by the radioactive substances.

The use of these radioactive tracers would appear to have a number of marked advantages. These stem from three facts: (1) radioactive compounds emit distinctive radiations that may be positively identified regardless of the chemical and physical composition of the fluid; (2) the identification of the radiation is quickly and easily accomplished—radiations of the gamma ray type can even be detected outside pipe or tank walls; (3) and the concentrations of radioactive material required for identification of flow are smaller than those required in usual chemical methods of flow measurement. Concentrations are so small that the density (and other fluid properties) is not appreciably altered by radioactive compounds in the requisite amounts. The alteration of density has heretofore been a problem in chemical flow tests and has limited the range of their application.

The first test planned would be along the lines of the Salt-Velocity test method. It will be performed with a small settling tank and the usual salt flow tests will be performed on the tank to determine its characteristics as measured with this medium. Then, in very nearly the same way, the radioactive material will be injected at the influent end of the tank and by placing the counter at the effluent end the passing of the radioactive material will be detected and measured and a mass-curve plotted. This curve can then be plotted and compared to the one obtained with the one from the salt flow method. We hope that the curve from the new method will give a much better measurement of the long tail of this curve and consequently a better measurement of the properties of the tank. This method could of course be used with sewage or any other liquid which is impossible with the salt test or any other chemical or colormetric methods. If this first test is successful then more will be performed.

Possibilities for this method of measurement seem to be unlimited if these tests show it to be practical. It could be used practically in turbine testing and in the design of integrating tanks for disposal of toxic industrial tanks, for example. Detention periods for all parts of a sewage treatment plant could be determined by this method. It could be adapted to hydraulic models to measure velocity, acceleration, direction, and the intermixing in two fluid systems (e.g. density flow in reservoirs and sewage field in tidal waters).

As no work has ever been attempted

in this field much preliminary work must be done. The selection of the correct material instruments and equipment are very important. Also doubly important is the necessary precautions that must be taken to insure that the health of the experimenter is not endangered by exposure to radiations.

It is hoped that this method will be found to be successful as a measuring device. This preliminary work is vital in proving its value one way or another and will perhaps lead others to the solution even if it is not found in this year of work.

Carroll T. Newton, Lt. Col. Corps of Engineers, U. S. Army, whose permanent address is Melrose, Massachusetts, was born in 1911 and graduated from the Massachusetts Institute of Technology with B.S. degree in Architectural Engineering in 1933 and M.S. degree in Civil Engineering in 1940. He served several years as Reserve Officer of various duties. He was commissioned 2nd Lieutenant in July 1937 and for two years worked on the Fort Peck Dam. Then, for two years he was Captain on duty with engineering troops until May, 1942. From June, 1942 to August, 1945 as Major he was Division Staff Officer in Charge of Logistics in California, North Africa, Italy and France. Later commanded an Engineer General Service Regiment in France with the rank of Colonel.

In 1945 he spent a year as Director of the U. S. Waterways Experiment Station, at Vicksburg, Mississippi. He was then entered as a student at the Armed Forces Staff College at Norfolk, Va. He is now instructor at the School of Logistics at Fort Leavenworth, Kansas.

Colonel Newton is a member of the Boston and American Societies of Civil Engineers and also the Society of American Military Engineers.

Colonel Newton will engage in a year's research in "Sediment Scour, Transportation and Decomposition". There are many complex problems of alluvial river control confronting public agencies, especially the Corps of Engineers, in the major waterways of the midwestern United States. Meander, bank caving, bar formation, and channel depth in alluvial rivers are of primary concern in control programs and involve sediment scour, transportation and deposition. The project will comprise study and analysis of the sediment carrying characteristics of water. It is expected the study will include some basic fluid mechanics, alluvial geology and investigation of the relation of suspended or moving particle size to velocity and density of sediment laden water.

The major portion of the scholarship year will be spent at a graduate school that can provide appropriate facilities and faculty direction; probably in the west or middle west. Coordination will be effected with related investigations, completed or being carried on by District and Division offices of the Corps of Engineers in the Mississippi and Missouri basins.

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETINGS

Boston Society of Civil Engineers

SEPTEMBER 22, 1948.—A regular meeting of the Boston Society of Civil Engineers was held this evening at the American Academy of Arts & Sciences, 28 Newbury Street, Boston, Mass.

President Weaver presided at the meeting and announced the following deaths:

William M. Smith who was elected a member February 20, 1925 and who died April 22, 1948.

C. Frank Allen who was elected a member March 24, 1875 an Honorary Member of the Society March 16, 1932 and who died June 6, 1948.

Pere O. Haak who was elected a member September 23, 1942 and who died July 12, 1948.

Hartley L. White who was elected a member May 15, 1895 and who died September 22, 1947.

Charles D. Kirkpatrick who was elected a member January 27, 1915 and who died August 3, 1948.

Frederic W. Bateman who was elected a member March 21, 1888 and who died September 2, 1948.

President Weaver announced that the October meeting of the Society would be omitted.

The Secretary announced that the following were elected to membership on September 20, 1948:

Grade of Student—Alvin M. Fine, Ralph R. Greene, Donald Kerivan, Logan T. Murdoch, Allan F. Platt, Arthur P. Rich, Frank T. Smith.

President Weaver announced that Honorary Membership in the Society had been conferred on one of the Society's distinguished members by vote of the Board of Government on March 24. 1948, to Prof. Dugald C. Jackson, who became a member of the Society on March 17, 1909. Prof. Dugald C. Jackson was unable to be present to accept the Certificate, due to illness, and on behalf of Prof. Jackson presentation of the Certificate was accepted by Prof. John B. Babcock, 3rd.

The Certificate read as follows:

BOSTON SOCIETY OF CIVIL ENGINEERS

In Recognition of His Outstanding Accomplishments as an Engineer and Educator in the Field of Electrical Engineering

DUGALD CALEB JACKSON
Has Been Duly Elected an
HONORARY MEMBER
By Direction of the Board of Government
March 24, 1948

Edwin B. Cobb — *Secretary*
Harvey B. Kinnison — *President*

(*Seal*)

President Weaver then introduced the speaker of the evening, Mr. Edward G. Lee, Assistant Designing Engineer, New England Power Service Company, who gave a most interesting talk on "The Proposed Redevelopment of the Water Power of the Connecticut River at Wilder, Vermont". The talk was illustrated with slides.

At the close of the meeting the

members gathered in the Lounge where a collation was served.

Meeting adjourned at 8:55 P.M.

One hundred five members and guests attended the meeting.

ROBERT W. MOIR, *Secretary*

HYDRAULICS SECTION

MAY 5, 1948.—A meeting of the Hydraulics Section was held at the Society Rooms on this date, following a dinner at the Smorgasbord.

The meeting was called to order at 7:20 P.M. by Chairman John G. W. Thomas who introduced the speaker of the evening, Professor James W. Daily of the Department of Civil and Sanitary Engineering, Massachusetts Institute of Technology. Professor Daily had as his subject, "Laboratory Investigations of the Mechanism of Cavitation." His lecture was based on a paper presented by Professor Robert T. Knapp, Director of the Hydrodynamics Laboratory of California Institute of Technology, before the American Society of Mechanical Engineers and which will appear in the Transactions of that Society. Professor Daily discussed the mechanism of cavitation and showed motion pictures and slides illustrating the formation and collapse of cavitation cavities. Following a discussion period a rising vote of thanks was extended by the 31 members and guests present.

The meeting adjourned at 8:40 P.M. and was followed by a brief meeting of the Executive Committee.

GARDNER K. WOOD, *Clerk*

APPLICATIONS FOR MEMBERSHIP

[July 1, 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 considerations. Communications relating to applicants are considered by the Board as strictly 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

HARRY L. KINSEL, Newtonville, Mass. (b. March 17, 1906, Altoona, Pennsylvania). B.S. in Sanitary Engineering, The Pennsylvania State College, June, 1928. Experience, June, 1928 to March, 1936, junior engineer with Arthur Richards, consulting engineer, at Altoona, Pa.; engineering assistant and later assistant engineer, Penna. Dept. of Health; assistant village engineer, Marmaroneck, N. Y., and designing engineer, Charles F. Mebus, consulting engineer, Glenside, Pa.; March, 1936 to March, 1940, assistant engineer with Greeley and Hansen, Chicago, on designs, investigations, and reports for water purification and sewage treatment works, pumping stations and special sewerage structures; including one year in charge of sewage treatment construction; March to December, 1940, assistant engineer with Metcalf & Eddy, Boston, on sewerage design and reports for metropolitan Boston; January to September 1941, assistant engineer (filter design), Bureau of Water, Philadelphia, Pa., in responsible charge of design of water filtration plant; September, 1941 to present date, project engineer in responsible charge of de-

signs, investigations, and reports relative to water supply, water purification, sewerage and sewage treatment. Refers to *E. S. Chase, E. F. Childs, A. J. Burdoin, F. S. Gibbs.*

HOWARD SIMPSON, Brookline, Mass. (b. March 30, 1922, New York, N. Y.). Received B.S.C.E. degree from Cornell University in 1942. Experience, from graduation to November, 1943, employed with O'Driscoll & Grove, Inc., of New York City, as engineer-in-training. Served at various times as time-checker, timekeeper, materialsman, assistant accountant, office engineer, transitman, and field engineer in connection with the construction of army camps and permanent housing. November, 1943, employed with Ford, Bacon & Davis, Inc., of New York City as instrumentman and assistant field engineer in the construction of a chemical plant. Job completed in May, 1944. With Kellex Corp. of New York City as engineering designer in connection with the construction of another chemical plant, leaving in July, 1944 to accept a commission in the Navy. Separated from the service in September, 1946. Commenced studying at Mass. Institute of Technology for the degree of S.M. in Building Engineering and Construction. The summer of 1947 worked at Jackson & Moreland of Boston as designer. September, 1947, started as instructor of structural design at Mass. Institute of Technology (incidentally receiving my S.M. degree in February, 1948). The past summer with Jackson & Moreland and am now returning to M.I.T. in the capacity of assistant professor of structural design. Refers to *A. G. H. Dietz, O. G. Julian, D. Peabody, Jr., C. H. Norris, W. C. Voss.*

Transfer from Grade of Junior

JOSEPH W. LAVIN, Bloomfield, N. J. (b. May 31, 1922, Dorchester, Mass.). Graduated from Northeastern University, November, 1943 with B.S. degree in Civil Engineering. Experience: November, 1943 to March, 1944, junior engineer, E. W. Branch., Inc., Quincy, Mass., office and field work on city surveying; March, 1944 to May, 1946, U. S. Army Corps of Engineers, 14 months of which was spent as project engineer on the rehabilitation of war damaged buildings in Manila, Philippine Islands; July, 1946 to June, 1947, assistant engineer, Chas. A. Maguire & Associates, Boston, Mass., on design of the East Boston Elevated Expressway; June, 1947 to date, principal assistant engineer, Edwards and Kelcey, Newark, N. J., on planning and design of urban freeways. Refers to *C. O. Baird, J. J. Cusack, E. A. Gramstorff, C. W. Newcomb.*

ALLAND L. LEVY, Palo Alto, California (b. March 17, 1921, Boston, Mass.). Graduated from Northeastern University in 1943, B.S. degree in Civil Engineering. Experience: 1943-1946, Corps of Engineers, U. S. Army; 1946-1947, assistant bridge engineer, State of California. 1947 to present, estimator and purchasing agent for Barrett & Hilp, general contractors of San Francisco. Refers to *C. O. Baird, E. A. Gramstorff, G. W. Hankinson, E. L. Spencer.*

For Admission

DONALD W. SCULLY, South Boston, Mass. (b. November 3, 1917, Brockton, Mass.) Graduated from Framingham High School in 1935; B.S. degree in Civil Engineering from Massachusetts Institute of Technology in 1939. Experience, 1939-1940, Metropolitan District Water Supply Commission, Junior Engineer; 1940, Federal Power Commission, Washington, D. C., Division of Power-Flood Control Surveys, Junior Engineer, Power Flood Control Studies; 1940, Ebasco Services Inc., New York, Cadet Civil Engineer, Hydroelectric Power Surveys and Studies, hydroelectric design and appraisals; 1940-1941, Stone & Webster Engineering

Corp., Boston, Mass., Designing Draftsman, reinforced concrete design, Industrial Buildings; 1941, Federal Works Agency, San Juan, P. R., Construction Superintendent, Airport Construction; 1941-1945, Civil Engineer Corps, U. S. Navy, rank, Lieut. Comdr. Assignments, Public Works, Shore Establishments, Runway Construction, Naval Barracks, Roads, Sea Walls. 1946, Massachusetts Institute of Technology Surplus Property Committee, Acquisition of Surplus Machine Tools for Institute; 1947, Self employed, Plant Layout, Machinery Processing, Plant Disposal; 1948, Designing Engineer, G. B. Cabot & Associates, Boston. Refers to *J. B. Babcock, 3rd, C. B. Breed, A. J. Bone, R. Newton Mayall.*

Transfer from Grade of Junior

DANIEL J. CONLIN, West Roxbury, Mass. (b. July 27, 1916, Boston, Mass.) Graduated from Northeastern University in 1940, with B.S. in Civil Engineering. Experience, July, 1940 to November, 1941, Civil Engineer with George A. Fuller Company of New York, on topographical and hydrographic surveys, pile design and inspection, superintendent of utilities and airport construction of the Naval Air Stations in Quonset Point, R. I., and Argentenia, Newfoundland; November, 1941 to January, 1943, employed by the McWilliams & Helmers Company of New York at their Arctic Base in Greenland as a Civil Engineer and Construction Superintendent. In these capacities designed and laid out roads, and airfields as well as supervised their construction; February, 1943 to June, 1947 served in the U. S. Naval Civil Engineer Corps with the rank of Lieut. Comdr. For two years, duty on the Solomon and Marianas Islands working on airfields, roads, malaria control hospitals, bridges, camp areas, oil farms, hangars, water towers, etc. Supervised the construction and design of a good many phases of the above named work. For the remaining two years served at U. S. Naval Air Station, Quonset, R. I., in the public works department. Had charge of snow removal, runway and road maintenance, transportation and heavy equipment, rigging, etc. At present Civil Engineer and Assistant Supt. of Construction, John Bowen Company, Boston. Refers to *E. A. Gramstorff, W. Grady, P. Levenson, C. O. Baird.*

PAUL D. KILLAM, Salem, Mass. (b. May 1, 1920, Salem, Mass.) Graduated from Northeastern University in 1943 with B.S. in Civil Engineering. Experience, cooperative work at the Engineering Dept. of Essex County. Served three years with the Army Air Force, Camp Engineer in charge of roads and utilities for a short time. In 1946, returned to the Engineering Dept. of Essex County where I now work as an Engineering Aide. Refers to *C. C. Barker, J. O. Harmaala, G. W. Hankinson, E. A. Gramstorff.*

ERNEST P. DEMERS, Middleton, Mass. (b. January 12, 1910, Wayland, Mass.) Graduated from Northeastern University, B.S.C.E., 1937; attended Brown University Evening School, special courses, advanced concrete and structural design, 1939; attended Massachusetts University Extension course in estimating at M.I.T., 1938; Centenary College evening school, Shreveport, Louisiana, business law course, 1946. Experience, 1930 to 1937, engaged concurrently while attending Northeastern University on cooperative plan as Rodman and Office Assistant in connection with general private practice survey work, including land court surveys, with K. E. McIntyre, Walpole, Mass. Same duties for Engineering Department, Walpole, Mass., during preliminary and construction survey work in connection with installation of new sewer system, and general municipal engineering; 1937 to 1941 engaged as contractor's field engineer on railroad crossing elimination work for Vermont

State Highway Department; on earth dam construction at Merriam Dam on Delaware Aqueduct Project for New York City Water Supply; on pressure pipe line construction for Boston, Mass., water supply for Metropolitan District Commission; 1939 to 1941 engaged as contractor's construction engineer on building construction. Duties of Chief Field Engineer on construction of hospital building and school building for City of Cambridge, Mass.; construction of low cost housing development for Fall River, Mass., Housing Authority; also Naval personnel housing development, Newport, Rhode Island; 1941 to 1946 engaged as construction engineer in various capacities, as Field, Office and Resident Engineer on industrial plant construction for Stone & Webster Engineering Corporation, Boston, Mass. At present Engineering Assistant in Construction Department, Stone & Webster Engineering Corporation, Boston Office, in connection with liaison and contractual work pertaining to construction activities. Refers to *H. T. Evans, W. S. Colby, C. M. Kelley, K. E. McIntyre.*

ADDITIONS

Members

Ethel H. Bailey, 30 Webster Place, Brookline 46, Mass.
William J. Balough, 5 Curve Street, Lexington 73, Mass.
Irving T. Berkland, 44 Cedar Street, Norwood, Mass.
Irving M. Brown, 45 Cleveland Street, Arlington 74, Mass.
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John S. Tewksbury, 77 Marion Road, Watertown, Mass.
Peter C. Tornabene, 372 Langley Road, Newton Center 59, Mass.
Louis W. Wise, Box 67, Lindsey Street, Attleboro, Mass.

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Ralph R. Greene, 10 Radford Place, Hyde Park 36, Mass.
Donald Kerivan, 2 Willow Street, Wellesley Hillls 82, Mass.
Logan T. Murdoch, 43 Newburg Street, Roslindale 31, Mass.
Allan F. Platt, 265 Beaver Street, Waltham, Mass.
Arthur P. Rich, 82 Summer Street, Stoneham, Mass.
Frank T. Smith, 865 V.F.W. Parkway, West Roxbury, Mass.

DEATHS

Frederic W. Bateman, September 2, 1948
Edwin J. Beugler, September 12, 1948
Pere O. Haak, July 12, 1948
Charles D. Kirkpatrick, August 3, 1948
William M. Smith, April 22, 1948
Hartley L. White, September 20, 1947

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