# LIVE-LOAD STRESSES

IN

# RAILWAY BRIDGES

WITH

## FORMULAS AND TABLES

BY

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# LIVE-LOAD STRESSES

## ARTICLE I.

### INFLUENCE LINES. DEFINITION AND USES.

INFLUENCE lines are useful in determining the position of live load on a bridge to produce maximum effect. They offer also a convenient method of deriving general algebraic formulas for stresses and rules for maximum when the general relations between influence lines and algebraic formulas are once understood; and in the case of the more complex problems of skew bridges, arches, cantilever bridges, etc., the influence lines themselves serve as a most direct method for the determination of the maximum live-load stresses.

An influence line may be defined as a line showing the variation in any function caused by a single *unit* load as it moves across the bridge. Vertical loads only will be considered. The function may be a reaction, bending moment, shear, stress, deflection, or any quantity whatsoever at a given part of a bridge, provided that its value is a function of the position of the unit load on the bridge.

Refer to Fig. 1a. Consider the span $AB$, and let $Z$ be any function at the fixed position $C$ on the span $L$. If the load unity moves across the span $AB$ and the value of $Z$ be calculated for each position of the unit load and its value $z$ plotted below the corresponding position of this load as an ordinate from a horizontal base line, the locus of the plotted points will be the influence line for $Z$. For example, if $Z$ be the bending moment at the fixed section $C$ in a beam of span $L$, the influence line will be as shown in Fig. 1b. In plotting influence lines, ordinates representing positive quantities are plotted above the base line; and negative, below. In case the influence line consists of several straight segments, it is necessary to determine the value of the ordinates only where the influence line has a change of direction; *i.e.*, at the *salient points*. For example,

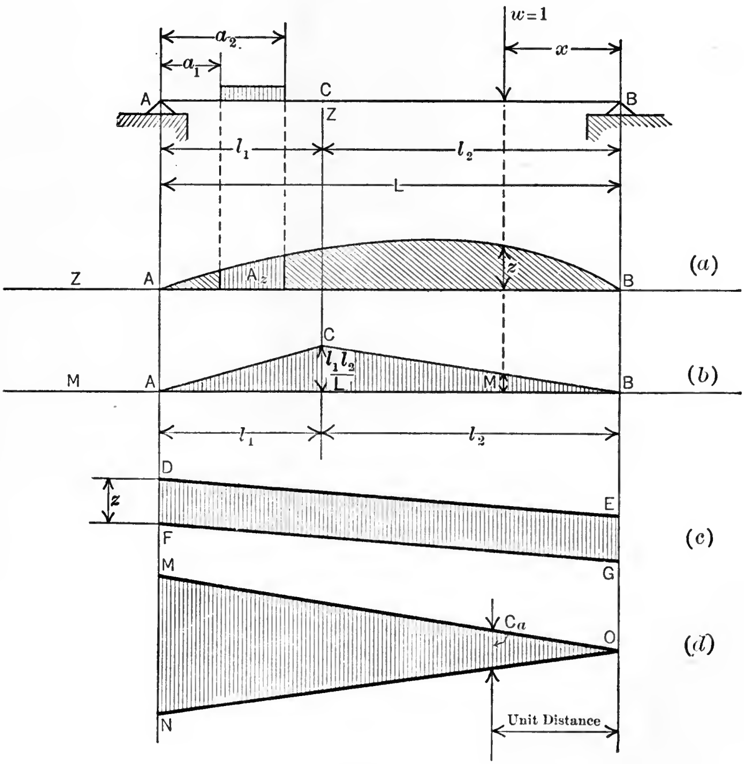


Fig 1.

the points $A$, $C$, and $B$ are the salient points of the influence line in Fig. 1b.

The value of $Z$ caused by a single load $w$ is equal to $wz$, if $z$ is the influence ordinate below $w$. The value of $Z$ caused by a series of loads $w_1$, $w_2$, $w_3$, etc., is

$$Z = w_1z_1 + w_2z_2 + w_3z_3 + \dots = \Sigma wz \quad . \quad . \quad . \quad (1)$$

where $z_1$, $z_2$, $z_3$, etc., are the influence ordinates below the corresponding loads. It will be convenient to speak of such a quantity as $wz$ as an *ordinate-load product.*

Formula (1) therefore may be expressed thus:

$Z$ = *Sum of ordinate-load products.*

The area between the influence line and the base line is called the *influence area.* It may be shown that the value of $Z$ caused by a uniform load on the bridge is proportional to the area $A_z$ of the influence line between the ordinates at the extremities of the uniform load. If the uniform load in Fig. 1a has an intensity of $q$ per unit of length, the load in the length $dx$ equals $q\,dx$, and the influence of this elementary load on the value of $Z$ is $zq\,dx$, where $z$ is the influence ordinate below $q\,dx$. Summing up for the length of the uniform load,

$$Z = q\sum_{a_1}^{a_2} z dx = qA_z \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

If a series of equal loads $w$ is on the span, the value of $Z$ is

$$Z = \Sigma wz = w\Sigma z \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

If a series of unequal loads, $w_1$, $w_2$, etc., is multiplied by the corresponding ordinates of an influence line or a portion of an influence line which has a constant ordinate $z$, as in Fig. 1c, the value of $Z$ is

$$Z = z(w_1 + w_2 + \ldots) = z\Sigma w = zW \quad . \quad . \quad . \quad (4)$$

where $W$ equals the sum of these loads.

If a series of unequal loads is multiplied by the corresponding ordinates of an influence line or a portion of an influence line consisting of two diverging lines, as shown in Fig. 1d, the value of $Z$, or the sum of the ordinate load products, and the rate at which $Z$ varies as the loading advances, are given by the two theorems that follow. The *slope* of a line is defined at the beginning of Art. 2.

*Theorem I.*

*The sum of the ordinate-load products* between two diverging lines equals the difference between the *slopes* of the two lines multiplied by the sum of the moments of the loads about the intersection of these lines.

In symbols, this is stated as

$$Z = C_a M_a \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

*Theorem II.*

The rate at which the sum of the ordinate-load products between the two diverging lines increases as the loading moves away from the intersection of these lines equals the difference between the *slopes* of the two lines multiplied by the sum of the loads.

In symbols, this is stated as

$$\frac{dZ}{dx} = C_a W_a = \frac{d(C_a M_a)}{dx} = C_a \frac{dM_a}{dx} \quad . \quad . \quad . \quad . \quad (5a)$$

The proofs of these theorems follow in the next article.

# ARTICLE II.

## SUM AND RATE OF VARIATION OF ORDINATE-LOAD PRODUCTS BETWEEN THE TWO DIVERGING LINES.

CONSIDER the diverging lines $DAB$ and $AC$ in Fig. 2. Use the following notation:

$w$ = any vertical load.

$z$ = ordinate below $w$ in the angle $BAC$.

$Z = \Sigma w_n z_n$ = sum of ordinate-load products.

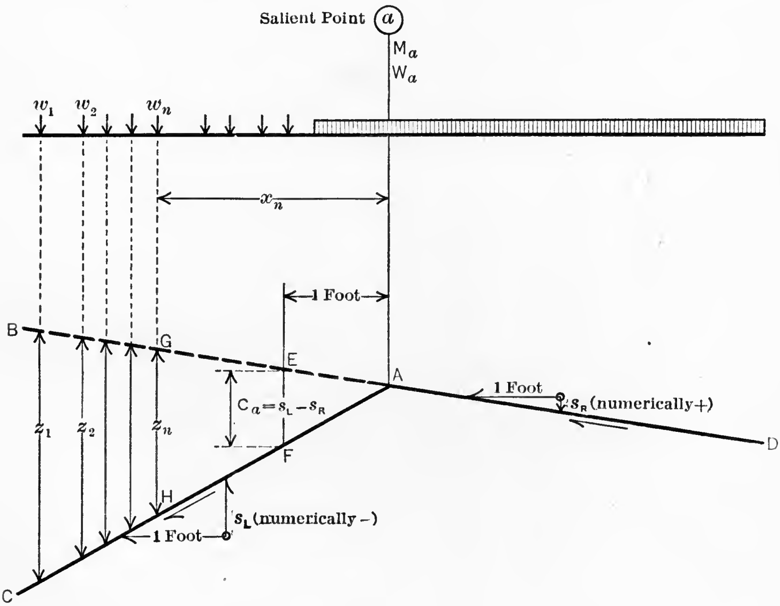


FIG. 2.

$M_a = \Sigma w_n x_n$ = moment sum of all loads to left of $Aa$ about $A$.

$W_a = \Sigma w_n$ = load sum of all loads to left of $Aa$.

$s_R$ = slope of line $DA$ = tangent of angle which $DA$ makes with the horizontal.

$s_L$ = slope of line $AC$ = tangent of angle which $AC$ makes with the horizontal.

$C_a = \frac{z_n}{x_n} = (s_L - s_R)$ = length of ordinate unit distance from $A$.

Slopes are counted numerically positive when upward to the left. The sign of $C_a$ (called the coefficient at salient point $A$) is, accordingly, negative when $AC$ diverges below $DA$ produced to the left of $A$. The value of $C_a$ may be determined graphically as $\frac{z_n}{x_n}$ or it may be figured algebraically as $(s_L - s_R)$.

*Proof of Theorem I, or that* $Z = C_a M_a$.

Consider the load $w_n$ distant $x_n$ from the salient point $a$. By the similar triangles $AEF$ and $AGH$,

$$\frac{C_a}{1.00} = \frac{z_n}{x_n}, \text{ or } z_n = C_a x_n.$$

Therefore,

$$w_n z_n = C_a w_n x_n. \quad \text{. . . . . . . . } (A)$$

Summing up all of the ordinate-load products,

$$Z = \Sigma w_n z_n = C_a \Sigma w_n x_n = C_a M_a. \quad \text{. . . . } (5)$$

*Proof of Theorem II, or that* $\frac{dZ}{dx} = C_a W_a$.

From equation ($A$) above, the increase in the ordinate-load product $w_n z_n$ for an advance $dx_n$ of the load is

$$w_n dz_n = C_a \cdot w_n \cdot dx_n.$$

Summing up the increases of all the ordinate-load products and noting that $dx$ is the same for all loads,

$$dZ = \Sigma w_n dz_n = C_a dx \cdot \Sigma w_n = C_a \cdot W_a \cdot dx.$$

Dividing by $dx$, $\frac{dZ}{dx} = C_a W_a = \frac{d(C_a M_a)}{dx} = \frac{C_a dM_a}{dx}$ . . . (5a)

## ARTICLE III.

### SUM AND RATE OF VARIATION OF ORDINATE-LOAD PRODUCTS FOR ANY INFLUENCE LINE. POSITION OF LOADING FOR MAXIMUM LIVE-LOAD STRESS.

An influence line of a general type is shown in Fig. 3, this one in particular being for the member $U_3L_4$ of the

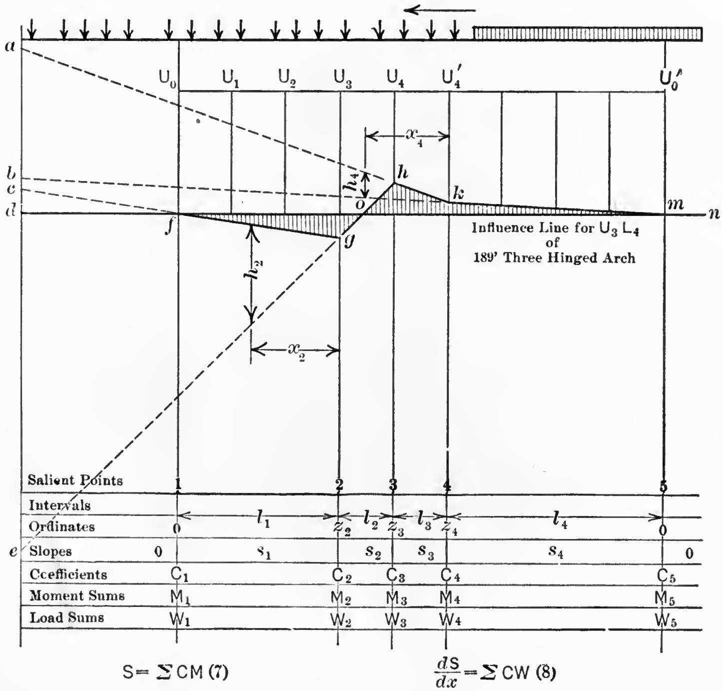


Fig. 3.

arch shown in Fig. 15. It is assumed that the ordinates at all salient points and the intervals between these points are known. Ordinates and slopes are counted positive or negative as already defined. The *slope of any segment* of the influence line equals the ordinate at the left minus the ordinate at the right end of this segment divided by the corresponding interval. The *coefficient C at any salient point* equals the slope of the segment at the left minus the slope of the segment at the right of this point. The subtractions in each case are made algebraically.

It should be remembered, as has already been pointed out in Art. 2, that the value of any coefficient $C$ may also be measured graphically from an influence line which has been drawn to scale. For example, in Fig. 3 the value of the coefficient $C_2 = \frac{h_2}{x_2}$ and $C_4 = \frac{h_4}{x_4}$.

The algebraic calculation of the coefficients at all salient points of the influence line in Fig. 3 is given below. If it be assumed that this influence line has been drawn to scale, the signs of the numerical values of the slopes and coefficients will be as given in the parentheses.

$$s_1 = \frac{0 - z_2}{l_1} \quad (+) \qquad C_1 = 0 - s_1 \quad (-)$$

$$s_2 = \frac{z_2 - z_3}{l_2} \quad (-) \qquad C_2 = s_1 - s_2 \quad (+)$$

$$s_3 = \frac{z_3 - z_4}{l_3} \quad (+) \qquad C_3 = s_2 - s_3 \quad (-)$$

$$s_4 = \frac{z_4 - 0}{l_4} \quad (+) \qquad C_4 = s_3 - s_4 \quad (+)$$

$$C_5 = s_4 - 0 \quad (+)$$

A numerical evaluation of the slopes and coefficients for this influence line is given in Fig. 15 of Art. 8, which the reader should check in order to understand completely the method of procedure. These coefficients should also be checked by the graphical method as already explained. For example, in Fig. 15 the value of $C_2 = \frac{2.59}{30} = .0863$.

It will be noted in the algebraic calculation of the coefficients $C$ at all salient points that each slope enters once as positive and once as negative. Therefore the sum of all coefficients equals zero.

$$\Sigma C = 0. \quad \ldots \ldots \ldots \quad (6)$$

This formula serves as a check on the values of the coefficients which have been determined either by calculation or by graphical measurement.

The general formulas for the sum of the ordinate-load products for any influence line (viz., with several salient points such as the one shown in Fig. 3) may be arrived at by considering the two contiguous sloping sides of the influence line meeting at each salient point as two diverging lines. The entire influence line is thus made up of pairs of diverging lines (see Fig. 3) to each pair of which formula (5) may be directly applied. Thus in Fig. 3,

| | | | |
|---|---|---|---|
| Ordinate-load products in | $\lfloor dfc$ | $= C_1M_1$ | $(-)$ |
| " " " " | $\lfloor cge$ | $= C_2M_2$ | $(+)$ |
| " " " " | $\lfloor eha$ | $= C_3M_3$ | $(-)$ |
| " " " " | $\lfloor akb$ | $= C_4M_4$ | $(+)$ |
| " " " " | $\lfloor bmd$ | $= C_5M_5$ | $(+)$ |

The signs of the $CM$'s are $+$ or $-$ according to the signs of the coefficients, for the $M$'s are always positive. Summing up the above equations and observing that the ordinate-load products cancel one another except between the influence line *fghkm* and its base line *fom*, it follows that the sum of the ordinate-load products for the influence line, or the live-load stress, is

$$S = C_1M_1 + C_2M_2 + \ldots = \Sigma CM. \quad \ldots \quad (7)$$

The letter $S$ represents in general any stress or sum of ordinate-load products for any influence line, while $Z$ stands for the sum of ordinate-load products for any geometrical figure.

The rate at which $S$ varies as the load advances a distance $dx$ equals

$$\frac{dS}{dx} = \frac{d(C_1M_1)}{dx} + \frac{d(C_2M_2)}{dx} + \text{Etc.}$$

But by formula (5a) this becomes

$$\frac{dS}{dx} = C_1W_1 + C_2W_2 + \ldots = \Sigma CW. \quad . \quad . \quad . \quad (8)$$

$W_1$, $W_2$, etc., = sum of all of the loads to the left of points 1, 2, etc., respectively, whether on the span or not.

$M_1$, $M_2$, etc., = moment of the same loads about points 1, 2, etc., respectively, whether on the span or not.

The above formulas (6), (7), and (8) apply equally well when the loading is headed from left to right instead of from right to left, the latter being the more usual way. In applying these formulas, however, it will save confusion not to reverse the loading, but to turn the influence line end for end, for this operation changes neither the values nor the signs of the coefficients $C$.

The stress $S = \Sigma CM$ is related to its derivative $\frac{dS}{dx} = \Sigma CW$ in the same way that any function is related to its derivative. Thus, if the value of $\frac{dS}{dx}$ passes through zero as the loading advances, the stress itself may have reached any one of four conditions; namely,

1. Numerically maximum positive value.
2. " minimum " "
3. " maximum negative "
4. " minimum " "

In practice it is desirable to find the positions of loading to satisfy the first and third conditions. This may be done by proceeding as directed below. It is assumed in stating the following rules that the live load is advancing from right to left. In case the live load advances from left to right, the wheel will be tried first to the left and then to the right of a salient point. In other words, $dx$ is always an increment in the same direction as the loading advances.

*Rule* 1.—To determine the position of loading to give a maximum positive stress, place the live load on the part of the bridge corresponding to the positive portion of the influence line. Try a wheel first immediately to the right of a salient point that has a *negative* coefficient and then just to the left of this point. Calculate the value of $\frac{dS}{dx} = \Sigma CW$ for each of these successive positions of loading. If the sign of $\frac{dS}{dx}$ changes from + to −, a position of loading for maximum positive stress is determined.

*Rule* 2.—To determine the position of loading to give a numerically maximum negative stress, place the live load on that part of the bridge corresponding to the negative portion of the influence line. Try a wheel first immediately to the right of a salient point that has a *positive* coefficient and then just to the left of this point. Calculate the value of $\frac{dS}{dx} = \Sigma CW$ for each of these successive positions of loading. If the sign of $\frac{dS}{dx}$ changes from − to +, a position of loading for numerically maximum negative stress is determined.

It will be noted that the negative coefficients $C$ occur at those salient points where the angles of the influence line point *upward,* while the positive coefficients $C$ occur at those salient points where the angles point *downward.*

It is unnecessary to seek a position of loading for *maximum positive* stress by placing a wheel successively to the right and to the left of any salient point which has a *positive* coefficient; for if $\frac{dS}{dx} = \Sigma CW$ be + when the wheel is to the right of this point, it would have a still larger +

value when the wheel is to the left of the point. A change, therefore, of $\frac{dS}{dx}$ from + to − would not result. Similarly, it may be shown to be unnecessary to seek a numerically *maximum negative* stress by trying wheels at any salient point which has a *negative* coefficient.

Formulas (7) and (8) are the general formulas for the solution of the sum of the ordinate-load products of an influence line and the rate of change of this sum, and are applicable to any form of influence line. They give at once a definite solution of the position of a set of loads producing maximum positive and negative stresses in any member of any truss or girder for which an influence line can be drawn and the values of such stresses. The method is particularly advantageous in the case of statically indeterminate structures, such as two-hinged and no-hinged arches, swing bridges, continuous girders, etc., where general analytical criteria for the positions of loads producing maximum stresses cannot readily be expressed and where such maximum stresses have had to be found by assuming positions of loadings and scaling the influence-line ordinates under all the loads, a laborious process and one open to much liability of mechanical inaccuracy.

In applying the present method to the simple forms of girders and trusses (viz., the statically determinate structures where the ordinates of the influence lines are readily expressible algebraically) it will generally be more convenient to transform formulas (7) and (8) in each case whereby the coefficients $C$ may be expressed in terms of the geometric proportions of the truss or girder. This, in the following articles (4 to 7 inclusive), we shall proceed to do for the case of girder bridges (with and without panels), pier reactions, and through Pratt trusses with curved or horizontal chords. The general method will, however, be applied *directly* to the case of the three-hinged arch in Art. 8, which will serve as a typical example of the application of the method to any influence line.

# ARTICLE IV.

## GIRDER BRIDGE WITHOUT PANELS.

IN Fig. 4 is shown a girder bridge without panels. The live load has advanced beyond the span, this being the

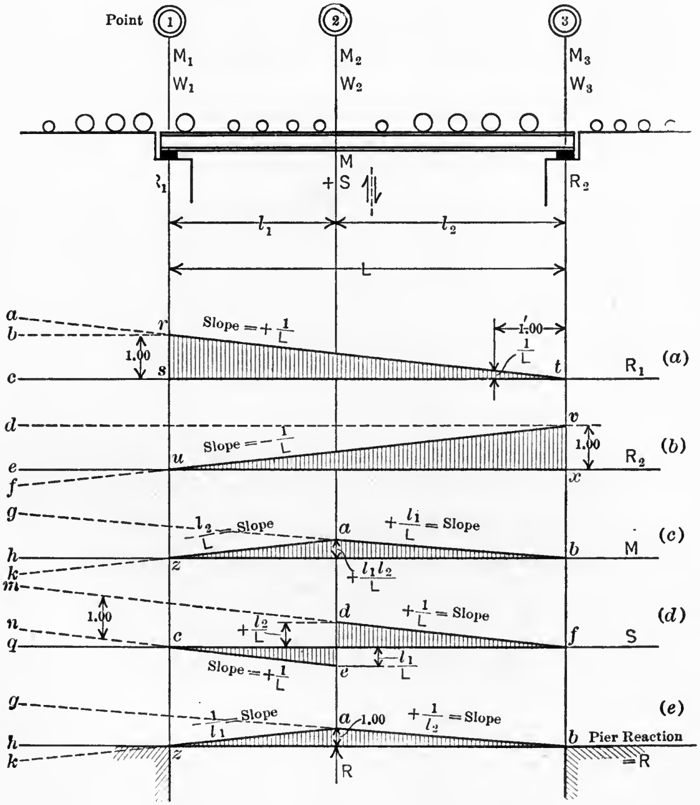


FIG. 4.

most general case. Formulas for the end reactions and for the bending moment and shear at any section will be developed.

The influence line for $R_1$ is shown in Fig. 4a. The sum of the ordinate-load products within the shaded area *rst* equals the end reaction $R_1$, which at the same time is the *end shear* at $R_1$.

From Fig. 4a,

| | | | | | |
|---|---|---|---|---|---|
| | Ordinate-load products in | | | | $|\underline{rst}$ = |
| | " | " | " | " | $|\underline{atc}$ |
| — | " | " | " | " | $|\underline{arb}$ |
| — | " | " | " | " | *brsc.* |

By using formulas (4) and (5), this equation becomes

$$R_1 = \frac{1}{L} M_3 - \frac{1}{L} M_1 - W_1 = \frac{M_3 - M_1}{L} - W_1 \quad . \; . \; (9)$$

Any value of $M$ or $W$ may be read directly from Table 2 for the standard loadings given in Table 1. For example, in Fig. 4, if $l_1 = 10'$, $l_2 = 30'$, and $w_1$ of Cooper's $E50$ has advanced 14′ beyond the left end of the span, we have from Table 2,

| | | |
|---|---|---|
| At 1, 14′ from $w_1$, | $M_1 = 350.0^{K'}$ | $W_1 = 62.50^{K}$ |
| At 2, 24′ from $w_1$, | $M_2 = 1150.0$ | $W_2 = 112.50$ |
| At 3, 54′ from $w_1$, | $M_3 = 5435.0$ | $W_3 = 177.50$ |

The formula for $R_2$ is developed as for $R_1$, the method of writing the second member of the first equation being abbreviated in a way readily understood. From the influence line in Fig. 4b, and the formulas (4) and (5),

$R_2$ = Ordinate-load products in (*dvxe* − $|\underline{dvf}$ + $|\underline{fue}$)

Or

$$R_2 = W_3 - \frac{1}{L} M_3 + \frac{1}{L} M_1 = W_3 - \frac{M_3 - M_1}{L} \quad . \; (9a)$$

The sum of the reactions $R_1$ and $R_2$ as given by (9) and (9a) equals $W_3 - W_1$, or the sum of the loads on the bridge.

From the influence line in Fig. 4c and formulas (5) or (7), the equation for bending moment may be written:

$M$ = Ordinate-load products in ( $|\underline{gbh}$ − $|\underline{gak}$ + $|\underline{kzh}$).

Or

$$M = \frac{l_1}{L} M_3 + \frac{l_2}{L} M_1 - M_2 \quad . \; . \; . \; . \; (10)$$

Formula (10) readily follows, likewise, from the general formula (7), $S = C_1M_1 + C_2M_2 + C_3M_3 = \Sigma CM$.

For example, in the case of the bending moment at point 2 in Fig. 4,

$$C_1 = 0 + \frac{l_2}{L}$$

$$C_2 = -\frac{l_2}{L} - \frac{l_1}{L} = -1$$

$$C_3 = \frac{l_1}{L} - 0$$

Whence $$M = \frac{l_2}{L} M_1 - M_2 + \frac{l_1}{L} M_3 \quad . \; . \; . \; . \; (10a)$$

Taking the derivative of $M$ with respect to the advance $dx$ of the loading toward the left or using formula (8) directly, the rate of variation of the bending moment is

$$\frac{dM}{dx} = \frac{l_1}{L} W_3 + \frac{l_2}{L} W_1 - W_2 \quad . \; . \; . \; . \; (11)$$

All positions for maximum $M$ may be found by trying wheels at point 2 as directed by Rule 1 of Art. 3. In applying this rule the simultaneous shifting of other wheels of the rigid loading from right to left of points 1 and 3 as a wheel is shifted from right to left of point 2, must be taken into account by substituting in formula (11) the corresponding changed values of $W_1$ and $W_3$. It is to be remembered, as stated in Art 3, that it is entirely unnecessary to try wheels at points 1 and 3.

From the influence line in Fig. 4d, the formula for the intermediate shear $S$ follows by applying formulas (4) and (5):

$S$ = Ordinate-load products in

$$(\lfloor\underline{mfq} - mden - \lfloor\underline{ncq})$$

Or

$$S = \frac{1}{L} M_3 - W_2 - \frac{1}{L} M_1 = \frac{M_3 - M_1}{L} - W_2 \quad . \quad (12)$$

There is one more thing to be borne in mind in calculating maximum bending moments in a girder bridge without panels: it is the rule for finding the section where the *absolute maximum bending moment* occurs. The rule is often spoken of as the "*centre of gravity rule,*" and may be stated as follows:

*The bending moment under any given wheel becomes maximum when the centre of the span bisects the distance from the wheel in question to the centre of gravity of the loading on the span.*

In the practical application of this rule, the procedure is first to find the wheel which gives maximum bending moment at the centre of the span and then to shift this wheel so that the bending moment beneath it becomes an absolute maximum according to the centre of gravity rule. For the usual standard loadings the maximum centre moment closely approximates the absolute maximum bending moment for the spans greater than 70 feet.

The proof of the centre of gravity rule follows. Refer to Fig. 5. Assume that it has been found by trial that the wheel $w_n$ gives the maximum centre moment. The general case where load has advanced beyond the span is taken. In order to get an *absolute maximum* bending moment under $w_n$, this wheel must be shifted a certain distance from the centre. Let such position be distance $y$ from $R_1$. The sum of the loads on the span is called $P_2$ and equals $(W_3 - W_1)$. The centre of gravity of the loads $P_2$ is distance $\bar{x}$ from $R_2$. The sum of the loads on the span to the left of $w_n$ is called $P_1$, and their centre of gravity is at the fixed distance $b$ from $w_n$.

Taking moments about $R_2$,

$$R_1 = \frac{P_2 \bar{x}}{L}$$

Therefore,

$$M = R_1 y - P_1 b = \frac{P_2 \bar{x}}{L} y - P_1 b.$$

In this equation for $M$, the only variables are $\bar{x}$ and $y$. Therefore, $M$ will be a maximum when the product $\bar{x}y$ is maximum. Note, however, that the sum

$$\bar{x} + y = (L - a) = \text{constant}.$$

If two variables have a constant sum, their product is maximum when the two variables are equal. Therefore, $M$ is maximum when $\bar{x} = y$. But when $\bar{x} = y$, the distance from $w_n$ to the centre of gravity of the loading is bisected

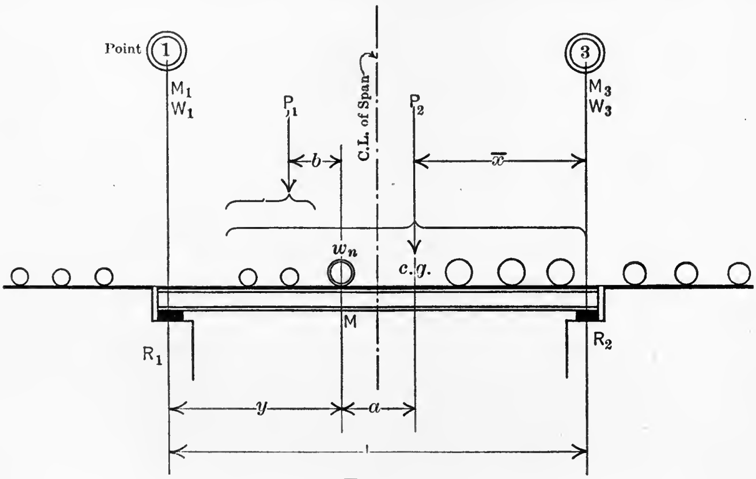


FIG. 5.

by the centre of the span. This proves the centre of gravity rule.

In order to apply this rule, a general expression for $\bar{x}$ is needed.

Since $R_1 = \dfrac{P_2\bar{x}}{L}$ it follows that $\bar{x} = \dfrac{R_1 L}{P_2}$. Substitute the value of $R_1$ from formula (9), and the value $(W_3 - W_1)$ for $P_2$.

$$\bar{x} = \frac{M_3 - M_1 - LW_1}{W_3 - W_1} \quad . \; . \; . \; . \; . \; . \; (13)$$

In the special case where the loading has not advanced beyond the left end of the span, $M_1$ and $W_1$ equal zero and $\bar{x}$ becomes

$$\bar{x} = \frac{M_3}{W_3} \quad \ldots \ldots \ldots \quad (13a)$$

Problems relating to a girder bridge without panels will now be given to illustrate the application of the above formulas and the use of some of the tables following the text.

*Problem.*—Given a 40-foot deck-girder bridge consisting of one girder per rail. Use Cooper's $E50$ loading. Find the maximum shear at the end, quarter point, and centre. Determine also the maximum bending moment at the quarter point and at the centre, and the absolute maximum bending moment. All values are to be given *per rail.*

*Solution.*—Table 5 following the text gives the position of Cooper's loadings for maximum end shear. This table is the result of the solution of end shears for a large number of spans. As a general rule, however, it is safe to assume that $w_2$ of Cooper's and similar loadings will always give the maximum end or intermediate shear when placed immediately to the right of the given section, the live load being headed toward the left. The exceptions in Table 5 to this general rule are not of prime importance, for the actual value of the shear when $w_2$ is used is sufficiently close to the maximum even in the exceptional cases. There is no satisfactory criterion for determining the position of loading for maximum shear in girder bridges without panels, for it is as easy to calculate the actual values of the shears for the successive positions of loading as it is to apply any criterion. In the case of bending moment, however, time is saved by using the criterion.

*Maximum End Shear.*

Use formula (9), $R_1 = \dfrac{M_3 - M_1}{L} - W_1$. Place wheel 2 of Cooper's $E50$ immediately to right of $R_1$. Take the values of moment and load sums for Cooper's $E50$ from Table 2.

$$\text{Maximum end shear} = \frac{4370 - 100}{40} - 12.5 = 94.3^k.$$

*Maximum Shear at Quarter Point.*

Use formula (12) with $w_2$ at quarter point.

$$S = \frac{M_3 - M_1}{L} - W_2$$

$$S \text{ at } \tfrac{1}{4} \text{ point} = \frac{2838.75 - 0}{40} - 12.5 = 58.5^k.$$

*Maximum Shear at Centre.*

Using formula (12) with $w_2$ at centre.

$$S \text{ at centre} = \frac{1600 - 0}{40} - 12.5 = 27.5^k.$$

The values for the shears are given in Kips, or thousand of pounds. A comparison of the above shears with those in Table 7 shows agreement of results.

*Maximum Bending Moment at the One-Quarter Point.*

First compute successive pairs of values for $\frac{dM}{dx}$ for different wheels, first placed to the right and then to the left of the quarter point. A change of sign from + to − indicates a wheel that gives a maximum. Use formula (11),

$$\frac{dM}{dx} = \frac{l_1}{L} W_3 + \frac{l_2}{L} W_1 - W_2 \quad . \quad . \quad . \quad . \quad (11)$$

$w_1$ at $\frac{1}{4}$ point.

$$\frac{dM}{dx} = \tfrac{1}{4}\,(112.5) + \tfrac{3}{4}\,(0) - 0 = +$$

No maximum.

$$\frac{dM}{dx} = \tfrac{1}{4}\,(112.5) + \tfrac{3}{4}\,(0) - 12.5 = +$$

$w_2$ at ¼ point.

$$\frac{dM}{dx} = \tfrac{1}{4}\,(145) + \tfrac{3}{4}\,(0) - 12.5 = +$$

Maximum.

$$\frac{dM}{dx} = \tfrac{1}{4}\,(145) + \tfrac{3}{4}\,(0) - 37.5 = -$$

$w_3$ at ¼ point.

$$\frac{dM}{dx} = \tfrac{1}{4}\,(145) + \tfrac{3}{4}\,(12.5) - 37.5 = +$$

Maximum.

$$\frac{dM}{dx} = \tfrac{1}{4}\,(161.25) + \tfrac{3}{4}\,(12.5) - 62.5 = -$$

$w_4$ at ¼ point.

$$\frac{dM}{dx} = \tfrac{1}{4}\,(161.25) + \tfrac{3}{4}\,(12.5) - 62.5 = -$$

No maximum.

$$\frac{dM}{dx} = \tfrac{1}{4}\,(177.5) + \tfrac{3}{4}\,(37.5) - 87.5 = -$$

Accordingly, compute the value of $M$ by formula (10) for $w_2$ and $w_3$ at quarter point.

$$M = \frac{l_1}{L} M_3 + \frac{l_2}{L} M_1 - M_2 \quad \ldots\ldots\ldots\ldots \quad (10)$$

$M$ for $w_2$ at quarter point,

$$M = \tfrac{1}{4}\,(2838.75) + \tfrac{3}{4}\,(0) - 100 = 609.7 \text{ Kip feet.}$$

$M$ for $w_3$ at quarter point,

$$M = \tfrac{1}{4}\,(3563.75) + \tfrac{3}{4}\,(37.5) - 287.5 = 631.6 \text{ Kip feet.}$$

The latter value, 631.6, is the maximum bending moment at the quarter point. A comparison of this value with Table 11 shows agreement of results. Reference to Table 3 indicates that the correct wheel for maximum has been chosen.

*Maximum Bending Moment at the Centre.*

$$\frac{dM}{dx} = \frac{W_3 + W_1}{2} - W_2, \text{ (10a), and}$$

$$M = \frac{M_3 + M_1}{2} - M_2, \text{ (11a), when } \frac{l_1}{L} = \tfrac{1}{2}$$

$w_3$ at centre,

$$\frac{dM}{dx} = \frac{128.75}{2} - 37.5 = +$$

No maximum.

$$\frac{dM}{dx} = \frac{128.75}{2} - 62.5 = +$$

$w_4$ at centre,

$$\frac{dM}{dx} = \frac{145}{2} - 62.5 = +$$

Maximum.

$$\frac{dM}{dx} = \frac{145}{2} - 87.5 = -$$

$w_5$ at centre,

$$\frac{dM}{dx} = \frac{145 + 12.5}{2} - 87.5 = -$$

No maximum.

$$\frac{dM}{dx} = \frac{161.25 + 12.5}{2} - 112.5 = -$$

Therefore, maximum centre moment occurs with $w_4$ at centre.

$$M = \frac{2838.75}{2} - 600 = 819.37 \text{ Kip feet.}$$

This value agrees with Table 11; and the position of loading, with Table 3.

*Absolute Maximum Bending Moment.*

Shift $w_4$ according to centre of gravity rule, and then recompute the value of $M$ under this wheel by formula (10). Note that new values for $l_1$, $l_2$, and $M_3$ must be determined.

By formula (13a), when $w_4$ is at the centre,

$$\bar{x} = \frac{M_3}{W_3} = \frac{2838.75}{145} = 19'.58$$

Therefore for absolute maximum bending moment under $w_4$, shift loading to left $\dfrac{20'.00 - 19'.58}{2} = 0'.21.$

The new values of $l_1$, $l_2$, and $M_3$ are

$$l_1 = 20.00 - 0.21 = 19.79$$
$$l_2 = 20.00 + 0.21 = 20.21$$
$$M_3 = 2838.75 + .21(145) = 2869.2$$

The absolute maximum bending moment =

$$M = \frac{l_1}{L} M_3 + \frac{l_2}{L} M_1 - M_2$$

$$= \frac{19.79}{40} (2869.2) + 0 - 600 = 819.54 \text{ Kip feet.}$$

It appears, therefore, that the absolute maximum bending moment is .17 Kip feet greater than the maximum centre moment. The difference is not great in this particular case, as the required shift of the loading is comparatively small. The position of loading for absolute maximum bending moment agrees with Table 4, and its value agrees with Table 7.

# ARTICLE V.

## PIER REACTION.

In Fig. 4e is given the influence line for the pier reaction $R$ between two non-continuous beam spans $l_1$ and $l_2$. From this influence line, the formulas (5) and (7) give

$R$ = Ordinate-load products in $(\underline{\lvert gbh} - \underline{\lvert gak} + \underline{\lvert kzh})$

Or,

$$R = \frac{M_3}{l_2} + \frac{M_1}{l_1} - \frac{L}{l_1\,l_2} M_2 = \frac{L}{l_1\,l_2}\left(\frac{l_1}{L} M_3 + \frac{l_2}{L} M_1 - M_2\right) \quad (14)$$

Formula (14) may also be derived from formula (10) since the ordinates of the influence line for $R$ bear the constant ratio $\frac{L}{l_1\,l_2}$ to the corresponding influence ordinates for $M$, the position of the live load and the values of $l_1$ and $l_2$ remaining fixed.

Therefore,

$$R = \frac{L}{l_1\,l_2} M \quad . \; . \; . \; . \; . \; . \; . \; . \; . \quad (16)$$

Substituting the value $M = \frac{l_1}{L} M_3 + \frac{l_2}{L} M_1 - M_2$ from formula (10) in formula (16), the result is again formula (14).

For equal spans,

$$l_1 = l_2 = l \text{ so that } R = \frac{M_3 + M_1 - 2M_2}{l} \quad . \quad (14a)$$

The rate of change of $R$ for a movement $dx$ of the loading to the left is

$$\frac{dR}{dx} = \frac{W_3}{l_2} + \frac{W_1}{l_1} - \frac{L}{l_1 l_2} W_2 = \frac{L}{l_1 l_2}\left(\frac{l_1}{L} W_3 + \frac{l_2}{L} W_1 - W_2\right) \quad (15)$$

For equal spans, $l_1 = l_2 = l$, so that

$$\frac{dR}{dx} = \frac{W_3 + W_1 - 2W_2}{l} \quad . \; . \; . \; . \; . \; . \; (15a)$$

In the last member of formula (15) the quantity within the parentheses is the same as the expression for $\frac{dM}{dx}$ in formula (11). It follows, therefore, that the same position of loading gives maximum $R$ and maximum $M$ for any given values of $l_1$ and $l_2$.

*Problem.*—(*a*) Find the maximum pier reaction per rail between two simple beam spans $l_1 = 10$ ft. and $l_2 = 30$ ft. (*b*) Find the maximum pier reaction between two simple beam spans, each having a length of 20 feet. Use Cooper's $E50$ loading.

*Solution of Problem* (*a*).

Use formula (15) to find position of loading for maximum $R$.

$$\frac{dR}{dx} = \frac{L}{l_1 l_2}\left(\frac{l}{L} W_3 + \frac{l_2}{L} W_1 - W_2\right) \quad . \; . \; (15)$$

$w_2$ at pier.

$$\frac{dR}{dx} = \frac{40}{10 \times 30}\left(\frac{10}{40}(145) + \frac{30}{40}(0) - 12.5\right) = +$$

Maximum.

$$\frac{dR}{dx} = \frac{40}{10 \times 30}\left(\frac{10}{40}(145) + \frac{30}{40}(0) - 37.5\right) = -$$

$w_3$ at pier.

$$\frac{dR}{dx} = \frac{40}{10 \times 30}\left(\frac{10}{40}(145) + \frac{30}{40}(12.5) - 37.5\right) = +$$

Maximum.

$$\frac{dR}{dx} = \frac{40}{10 \times 30}\left(\frac{10}{40}(161.25) + \frac{30}{40}(12.5) - 62.5\right) = -$$

Use formula (14) to compute the value of $R$.

$$R = \frac{M_3}{l_2} + \frac{M_1}{l_1} - \frac{L}{l_1 l_2} M_2.$$

$w_2$ at pier.

$$R = \frac{2838.75}{30} + \frac{0}{10} - \frac{40}{10 \times 30}(100) = 81^k.$$

$w_3$ at pier.

$$R = \frac{3563.75}{30} + \frac{37.5}{10} - \frac{40}{10 \times 30}(287.5) = 84^k.$$

The latter value of $84^k$ is the maximum pier reaction. Its value agrees with Table 14 and the position of loading agrees with Table 3.

*Solution of Problem (b).*

Use formulas (14a) and (15a),

$$R = \frac{M_3 + M_1 - 2M_2}{l}, \text{ and } \frac{dR}{dx} = \frac{W_3 + W_1 - 2W_2}{l}.$$

$w_3$ at pier.

$$\frac{dR}{dx} = \frac{128.75 + 0 - 2 \times 37.5}{20} = +$$

No maximum.

$$\frac{dR}{dx} = \frac{128.75 + 0 - 2 \times 62.5}{20} = +$$

$w_4$ at pier.

$$\frac{dR}{dx} = \frac{145 + 0 - 2 \times 62.5}{20} = +$$

Maximum.

$$\frac{dR}{dx} = \frac{145 + 0 - 2 \times 87.5}{20} = -$$

$w_5$ at pier.

$$\frac{dR}{dx} = \frac{145 + 12.5 - 2 \times 87.5}{20} = -$$

No maximum.

$$\frac{dR}{dx} = \frac{161.25 + 12.5 - 2 \times 112.5}{20} = -$$

Therefore, maximum pier reaction occurs when $w_4$ is at the pier.

$$R = \frac{2838.75 - 0 - 2 \times 600}{20} = 81.9^k.$$

This maximum pier reaction of $81.9^k$ agrees with value in Table 7 and Table 14, while the position of loading agrees with that given by Table 3.

# ARTICLE VI.

## GIRDER BRIDGE WITH PANELS.

In Fig. 6 is shown a girder bridge with panels. It is as-

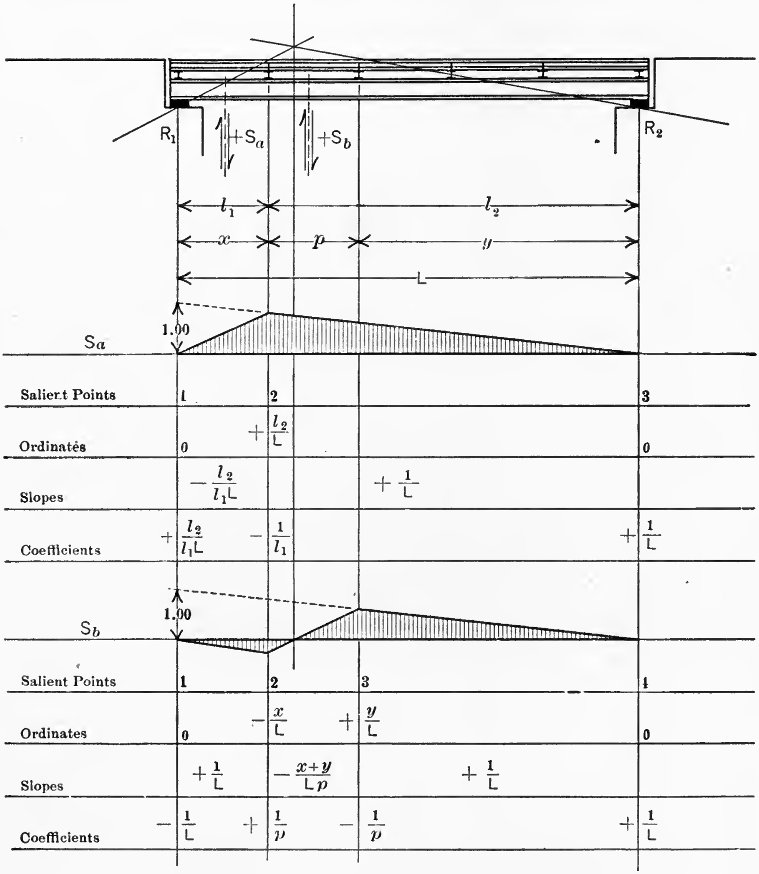


FIG. 6.

sumed that the live load has advanced beyond the left end of the span, this being the most general case.

The formulas for $R_1$ and $R_2$ are the same as formulas (9) and (9a) for the girder without panels, if the girder bridge with panels has end floor-beams; but if this bridge has end struts with the end stringers resting on separate pedestals, the value of $R_1$ beneath the end of the main girder is the same as $S_a$, the shear in the end panel, as given by formula (17) to follow.

Inasmuch as the maximum bending moment in a beam carrying concentrated loads always occurs beneath a concentration, the maximum bending moments in the main girder of a girder bridge with panels will occur at the floor-beams. The influence line for the bending moment at the floor-beams is the same as for the bending moment in a girder bridge without panels; accordingly, formulas (10) and (11) are to be used in finding maximum bending moments at the floor-beams.

It remains to derive formulas for the maximum shears $S_a$ in the end panel and $S_b$ in any intermediate panel. In Fig. 6 are given the influence lines for $S_a$ and $S_b$. The correctness of the ordinates is at once evident. The slopes and coefficients are calculated as explained in Arts. 2 and 3. The general formulas for $S_a$ and $S_b$ and their rates of variation may be written at once by use of formulas (7) and (8).

$$S_a = \frac{1}{L}M_3 + \frac{l_2}{l_1L}M_1 - \frac{1}{l_1}M_2 = \frac{1}{l_1}\left(\frac{l_1}{L}M_3 + \frac{l_2}{L}M_1 - M_2\right) \quad (17)$$

$$\frac{dS_a}{dx} = \frac{1}{L}W_3 + \frac{l_2}{l_1L}W_1 - \frac{1}{l_1}W_2 = \frac{1}{l_1}\left(\frac{l_1}{L}W_3 + \frac{l_2}{L}W_1 - W_2\right) \quad (18)$$

$$S_b = \frac{1}{L}M_4 - \frac{1}{p}M_3 + \frac{1}{p}M_2 - \frac{1}{L}M_1 \quad . \; . \; . \; . \; . \; . \; . \; . \quad (19)$$

$$\frac{dS_b}{dx} = \frac{1}{L}W_4 - \frac{1}{p}W_3 + \frac{1}{p}W_2 - \frac{1}{L}W_1 \quad . \; . \; . \; . \; . \; . \; . \quad (20)$$

Formula (17) when compared with formula (10) shows that $S_a$ is equal to the bending moment at the first intermediate floor-beam divided by the length of the first panel. Formula (18) when compared with formula (11) shows that the same position of loading that gives maximum bending moment at the first intermediate floor-beam will also give maximum shear in the end panel.

Formulas (19) and (20) are perfectly general and will serve for any assumed series of vertical loads in any position. For the usual standard loadings and panel lengths, however, it is not necessary to advance any loads beyond an intermediate panel for maximum shear in this panel. Therefore, for practical purposes formulas (19a) and (20a)

$$S_b = \frac{M_4}{L} - \frac{M_3}{p} = \frac{1}{p}\left(\frac{p}{L} M_4 - M_3\right) \quad . \quad . \quad (19a)$$

$$\frac{dS_b}{dx} = \frac{W_4}{L} - \frac{W_3}{p} = \frac{1}{p}\left(\frac{p}{L} W_4 - W_3\right) \quad . \quad . \quad (20a)$$

*Illustrative Problem.*—A single track through girder bridge with a floor system consisting of stringers and floor-beams, both end and intermediate, has six panels of 20 feet each. Find the maximum end reaction and the shear in panels $0 - 1$, $1 - 2$, and $2 - 3$, using Cooper's $E50$ loading.

*Solution.*—For maximum end reaction place wheel 2 at left end. Use formula

$$R_1 = \frac{M_3 - M_1}{L} - W_1 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

$$R_1 = \frac{27651 - 100}{120} - 12.5 = 217.1^k$$

Note that the above value agrees with Table 7.

For maximum shear in panel $0 - 1$, find critical wheel by formula (18) and then compute shear by formula (17).

Try wheel 3 at panel point 1.

$$\frac{dS_a}{dx} = \frac{1}{20}\left(\frac{1}{6}(365) + 0 - 37.5\right) = +$$

Maximum.

$$\frac{dS_a}{dx} = \frac{1}{20}\left(\frac{1}{6}(365) - 0 - 62.5\right) = -$$

Note that the position of loading agrees with Table 3. For this position of loading formula (17) gives

$$S_a = \frac{1}{20}\left(\frac{1}{6}(21895) + 0 - 287.5\right) = 168.1^k.$$

For maximum shears in the intermediate panels, determine the position of loading by formula (20a) and the shear by formula (19a).

$$\frac{dS_b}{dx} = \frac{1}{p}\left(\frac{p}{L}W_4 - W_3\right) \quad . \; . \; . \; . \; . \; . \; . \; . \quad (20a)$$

$$S_b = \frac{1}{p}\left(\frac{p}{L}M_4 - M_3\right) \quad . \; . \; . \; . \; . \; . \; . \; . \quad (19a)$$

Panel 1–2. Try wheel 3 at panel point 2.

$$\frac{dS_b}{dx} = \frac{1}{20}\left(\frac{1}{6}(306.25) - 37.5\right) = +$$

Maximum.

$$\frac{dS_b}{dx} = \frac{1}{20}\left(\frac{1}{6}(322.50) - 62.5\right) = -$$

$$S_b = \frac{1}{20}\left(\frac{1}{6}(15051.25) - 287.5\right) = 111.0^k.$$

Panel 2–3. Try wheel 3 at panel point 3.

$$\frac{dS_b}{dx} = \frac{1}{20}\left(\frac{1}{6}(240) - 37.5\right) = +$$

Maximum.

$$\frac{dS_b}{dx} = \frac{1}{20}\left(\frac{1}{6}(240) - 62.5\right) = -$$

$$S_b = \frac{1}{20}\left(\frac{1}{6}(9345) - 287.5\right) = 63.5^k.$$

The above values for shears agree with the values given by Table 9. The wheel for maximum shear in panels of girder and truss bridges is given in Table 6.

# ARTICLE VII.

## THROUGH PRATT TRUSS. GENERAL FORMULAS FOR LIVE-LOAD STRESSES AND THEIR RATE OF VARIATION. ILLUSTRATIVE PROBLEMS.

THE general formulas $S = \Sigma CM$ and $\dfrac{dS}{dx} = \Sigma CW$ may be used to write the equations for the live-load stresses in any member of a framed structure as soon as its influence

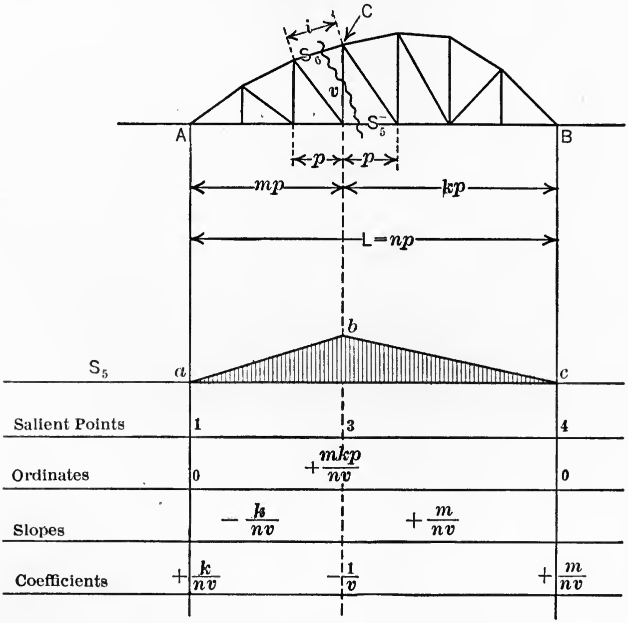


FIG. 7.

line has been drawn and the ordinates at the salient points determined.

In Figs. 7, 8, 9, and 10 are shown all the influence lines needed in writing the formulas for the live-load stresses in a through Pratt truss with non-parallel or parallel chords. The influence ordinate at any salient point is the calculated stress due to a one-pound load on the bridge at the panel point above this salient point. By easily discovered relations between similar triangles, the algebraic value of each stress, or influence ordinate, is expressed in terms that are most readily evaluated in any numerical problem.

The derivation of any one formula for a live-load stress is typical. Refer to Fig. 7. The stress in the lower chord member $S_5$ is found by taking moments about $C$. The influence line for $S_5$ is straight over each of the two intervals $kp$ and $mp$. The ordinates at the salient points 1 and 4 are zero. The ordinate at salient point 3 must be found by placing a one-pound load at the lower panel point of the truss above this salient point and calculating the value of $S_5$. For the unit load so placed,

$$\text{Reaction at } A = \frac{kp}{np} = \frac{k}{n}$$

By moments about $C$,

$$\frac{k}{n}(mp) = S_5 (v)$$

Therefore,

$$S_5 = + \frac{mkp}{nv} = \text{Influence ordinate at 3.}$$

The slopes of the segments of this influence line follow.

$$\text{Slope of } ab = - \frac{mkp}{nv} \div mp = - \frac{k}{nv}$$

$$\text{Slope of } bc = + \frac{mkp}{nv} \div kp = + \frac{m}{nv}$$

The coefficients $C$ for use in the general formula $S = \Sigma CM$ are now found.

$$C_1 = 0 + \frac{k}{nv} = + \frac{k}{nv}$$

$$C_3 = -\frac{k}{nv} - \frac{m}{nv} = -\frac{1}{v}$$

$$C_4 = \frac{m}{nv} - 0 = +\frac{m}{nv}$$

Therefore, for the position of the live load advanced beyond the limits of the span, the general formula for $S_5$ is

$$S_5 = \left(\frac{m}{nv}\right)M_4 - \left(\frac{1}{v}\right)M_3 + \left(\frac{k}{nv}\right)M_1.$$

However, in actual practice it is usually not necessary to advance the loading beyond the left end of the span in order to get a maximum value of $S_5$. The usual formula will therefore not contain the term $M_1$, since this will be zero; thus,

$$S_5 = \left(\frac{m}{nv}\right)M_4 - \left(\frac{1}{v}\right)M_3 \quad . \quad . \quad . \quad . \quad . \quad (21)$$

Inasmuch as the horizontal component of the stress $S_6$ in an inclined top chord member or end post equals the stress $S_5$ in a corresponding lower chord member, the stress $S_6$ in any top chord member or end post may be found by

$$S_6 = \frac{i}{p} \cdot S_5 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (22)$$

In Fig. 8 is shown the influence line for the stress $S_4$ in any vertical post. The influence ordinates are determined by taking moments about the intersection of the upper and lower chord members which are cut by the section. The algebraic values of these ordinates are transformed by use of easily discovered relations between similar triangles. The slopes and coefficients are then calculated in the usual way. The influence line indicates that the live load should advance into but not beyond the panel $p$ for a maximum compression, and for this reason $M_1$ and $M_2$ equal zero for the usual case. The numerical value of the maximum *compression* $S_4$ in a vertical post is, therefore,

$$S_4 = \left(\frac{a}{bL}\right)M_4 - \left(\frac{1}{p}\right)M_3 \quad . \quad . \quad . \quad . \quad . \quad (23)$$

The coefficients for the stress in any inclined web member are given by Fig. 9. The quantities for $S_1$ and $S_2$ are

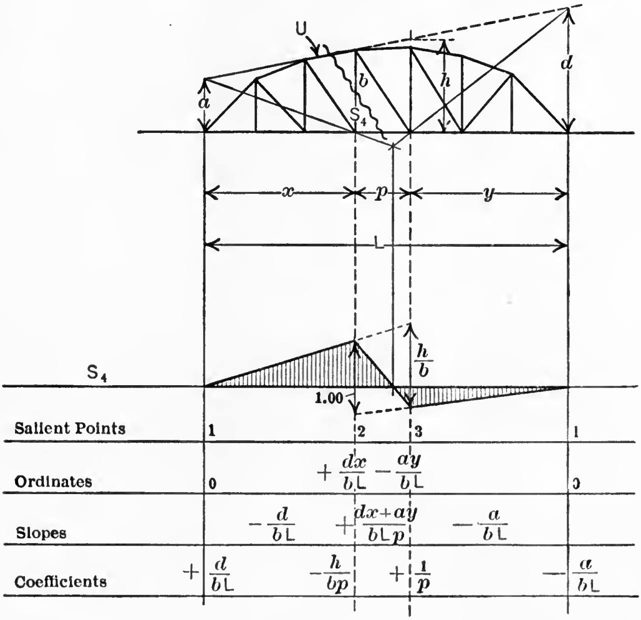


FIG. 8.

as shown, and the quantities for $S_3$ are of the same algebraic form except that they are of opposite sign throughout. For the usual position of the live load advanced from the right into but not beyond the panel $p$ for maximum stress, the moment sums $M_1$ and $M_2$ equal zero, and the numerical values of the maximum tension $S_1$ and $S_2$ and of the maximum compression $S_3$ are given by the following formula:

$$S_1,\ S_2,\text{ or } S_3 = \left(\frac{ta}{cbL}\right)M_4 - \left(\frac{t}{bp}\right)M_3 \quad . \quad . \quad . \quad (24)$$

In a special case where the loading must be advanced beyond the panel $p$ until the tension in the inclined counter-web member $S_2$ is balanced by the dead-load compression

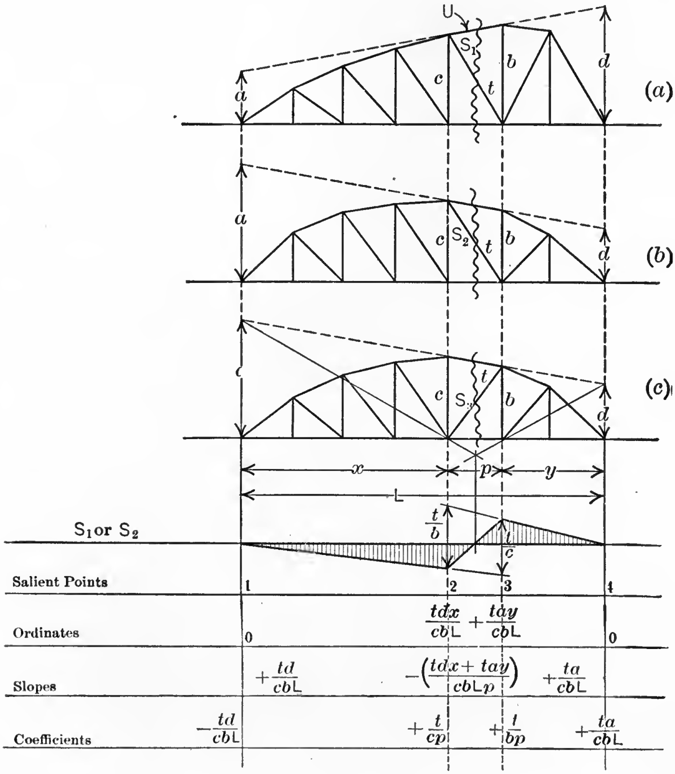


FIG. 9.

in this same member, the value of $M_2$ is not zero, and the formula for $S_2$ becomes

$$S_2 = \left(\frac{ta}{cbL}\right)M_4 - \left(\frac{t}{bp}\right)M_3 + \left(\frac{t}{cp}\right)M_2$$

$$\text{Or, letting } M_c = \left(M_3 - \frac{b}{c}M_2\right),$$

$$S_2 = \left(\frac{ta}{cbL}\right)M_4 - \frac{t}{bp}\left(M_3 - \frac{b}{c}M_2\right) = \left(\frac{ta}{cbL}\right)M_4 - \left(\frac{t}{bp}\right)M_c \quad (25)$$

Note that the coefficients of $M_4$ and $M_c$ in this formula are the same as the coefficients for $M_4$ and $M_3$ in formula (24).

The influence line for the counter-tension in a vertical post is shown in Fig. 10. For the usual case, the loading advances beyond the panel but not beyond the end of the span. Therefore $M_1$ is equal to zero, so that

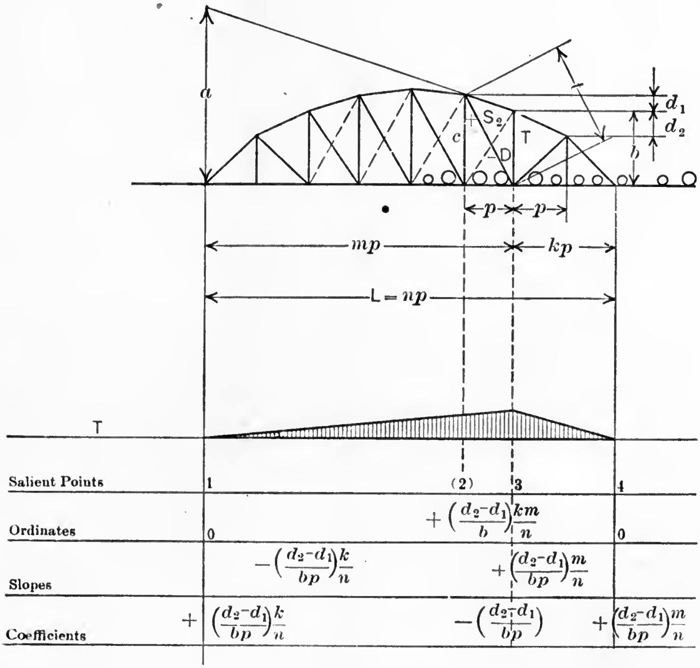


FIG. 10.

$$T = \left(\frac{d_2 - d_1}{bp}\right)\left(\frac{m}{n} M_4 - M_3\right) = K \cdot M_o \quad . \quad . \ (26)$$

where $K$ and $M_0$ stand for the corresponding terms in the parentheses. In order that $T$ be a maximum the live load must advance beyond the position for the maximum tension $S_2$ until the tension as computed by formula (25) becomes equal to the dead-load compression in this same member. For this position of the live load, the value of $T$ is then computed by using formula (26). It may be noted that some specifications state that only ⅔ of the dead-load compression is to be counted as effective in counteracting the live-load tension in an inclined counter-web member. This specification has been observed in the problem to follow.

A review of the preceding formulas shows that all the live-load stresses may be computed by formulas (21), (22), (23), and (24), except the counter-tension in a vertical post and the tension in a floor-beam hanger. Formula (25) makes it possible to find readily by trial the position of loading for maximum counter-tension in a vertical post, and formula (26) gives the value of this tension. The maximum tension in the floor-beam hanger may be found by the use of formulas (14a) and (15a) for pier reaction between equal spans.

If the chords of the Pratt truss are parallel, there will be no counter-tension in any vertical post. Formula (21) for the stress in a horizontal chord member and formula (22) for the stress in the inclined end post remain unchanged. Formulas (23) and (24) for web stresses are simplified because $a = b =$ depth of truss.

The formulas, therefore, for the Pratt truss with parallel chords are:

Stress in horizontal chord members =

$$S_5 = \left(\frac{m}{nv}\right)M_4 - \left(\frac{1}{v}\right)M_3 \quad . \quad . \quad . \quad . \quad . \quad . \quad (21)$$

Stress in inclined end post $= S_6 = \frac{i}{p} S_5$ . . . . . . . . (22)

Stress in vertical post $= S_4 = \left(\frac{1}{L}\right)M_4 - \left(\frac{1}{p}\right)M_3$ . . . (29)

Stress in inclined web member =

$$S_1 = \left(\frac{t}{cL}\right)M_4 - \left(\frac{t}{cp}\right)M_3 = \frac{t}{c} S_4 \quad . \quad . \quad . \quad . \quad (30)$$

One general formula will suffice for finding the position of loading for maximum chord and web stresses of a Pratt truss with either inclined or parallel chords. The formulas

(21), (23), (24), (29), and (30) for these stresses are of one general form

$$S = (G)\,M_4 - (H)\,M_3 \quad . \quad . \quad . \quad . \quad . \quad (27)$$

where $G$ and $H$ are the corresponding coefficients of $M_4$

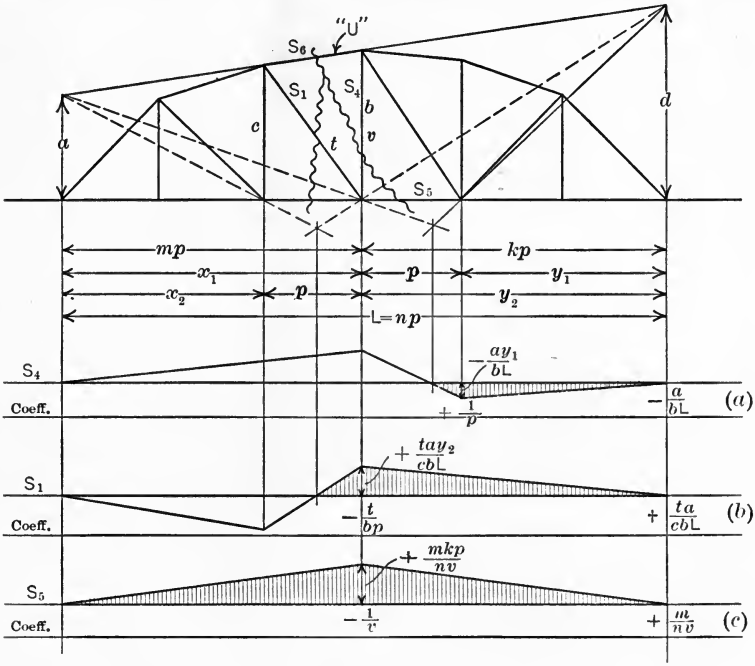


FIG. 11.

and $M_3$ in the preceding formulas. The rate of variation of $S$ as the load advances is

$$\frac{dS}{dx} = GW_4 - HW_3 = H\left(\frac{G}{H}W_4 - W_3\right) \quad . \quad . \quad (28)$$

When any one of the above stresses is a maximum, the value of $\left(\frac{G}{H}W_4 - W_3\right)$ passes through zero as a wheel is shifted from right to left of the salient point 3 in Figs. 7, 8, or 9.

The preceding formulas for the live-load stresses are summarized for convenient reference in Art. 11 preceding the Tables. The important dimensions and quantities in Figs. 7, 8, and 9 are summarized in Fig. 11. If a uniform live load is used, the shaded areas in Fig. 11a, b and c multiplied by the intensity of the uniform load will give the maximum live-load stresses. The algebraic value of any one of these triangular areas is conveniently expressed as the base of the triangle times ½ of the given algebraic ordinate. The lengths of the bases of the shaded areas in Figs. 11a and b may be readily determined by one of the constructions shown in Figs. 12a and 12b, which give the position of the unit load for zero stress in the members indicated. The proofs that these constructions locate neutral points are not given, for they are generally known, and are proved in numerous texts on bridges. (See Marburg's "Framed Structures and Girders," Vol. I, page 392.)

The application of the preceding formulas will now be made to the calculation of the live-load stresses in the two single track through Pratt trusses shown in Figs. 13 and 14. A convenient procedure is as follows:

1. Determine the lengths of all inclined members and write their values on the truss outline.

2. Determine the values of the intercepts $a$ as defined by Fig. 11 and write their values on the truss outline.

3. Write on the truss outline the distances of the several panel points from the right end of the span.

4. Write down the reciprocals of the span, panel length, and lengths of vertical members.

5. Make a form for tabulating calculations and list members in some convenient form as is done in Figs. 13 and 14.

6. Calculate the numerical values of the coefficients $G$ and $H$ for the several members by use of the formulas already derived.

7. Determine the position of the loading for maximum stress by finding the position of loading causing $\left(\frac{G}{H}W_4 - W_3\right)$

to pass through zero, and for this position of loading select from Table 2 the corresponding values of $M_4$ and $M_3$. At

VARIOUS CONSTRUCTIONS USED TO FIND NEUTRAL POINTS IN PRATT TRUSSES.

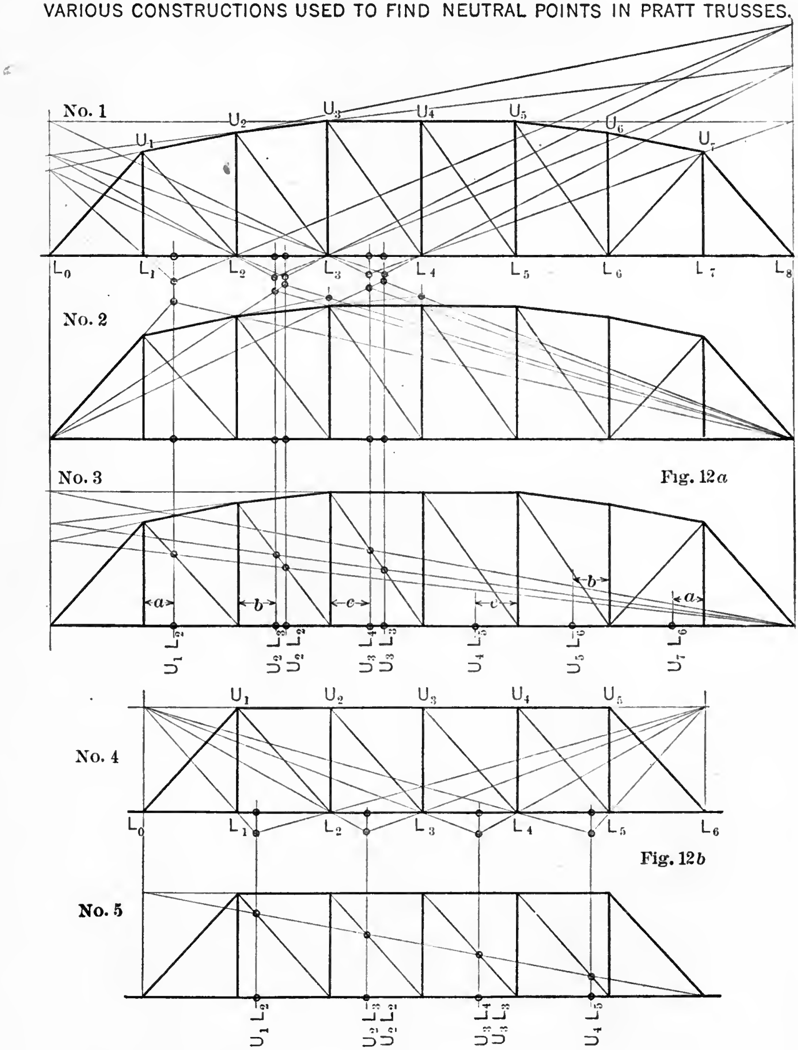


FIG. 12.

the same time tabulate the length $L_1$ of loading causing maximum stress as this value is used in the impact formula

$$I = S \cdot \frac{300}{L_1 + 300}.$$

8. Calculate values of $S = GM_4 - HM_3$ and combine with impact and dead-load stresses. When the dead- and live-load stresses are of opposite sign, the combination is usually not algebraic but according to the particular specification that is used.

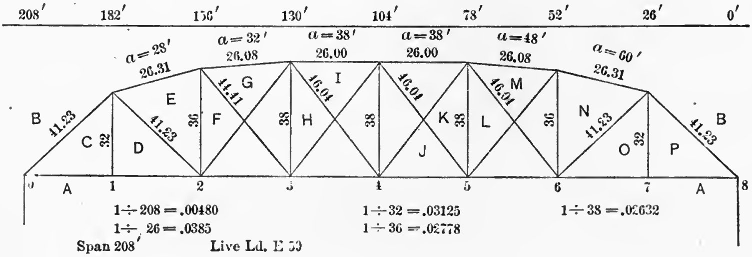


FIG. 13.

| Mem. | G | H | Wheel | $M_4$ | $M_3$ | $GM_4$ | $HM_3$ | S | $L_1$ | $\frac{300}{L_1+300}$ | I | DL | Total K |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| EF | .00373 | .0385 | 3 @ 3 | 33970 | 287 | 127 | 11 | −116 | 143 | .677 | − 78 | − 40 | −234 |
| ED | .00481 | .0442 | 3 @ 2 | 46255 | 287 | 223 | 13 | +210 | 169 | .640 | +134 | + 83 | +427 |
| GH | .00405 | .0385 | 2 @ 4 | 21531 | 100 | 87 | 4 | − 83 | 112 | .728 | − 60 | − 15 | −158 |
| GF | .00500 | .0450 | 3 @ 3 | 33970 | 287 | 170 | 13 | +157 | 143 | .677 | +106 | + 48 | +311 |
| IJ | .00480 | .0385 | 2 @ 5 | 12940 | 100 | 62 | 4 | − 58 | 86 | .777 | − 45 | + 7 | ..... |
| IH | .00580 | .0466 | 3 @ 4 | 23375 | 287 | 136 | 13 | +123 | 117 | .719 | + 88 | + 21 | +232 |
| JK | .00580 | .0466 | 2 @ 5 | 12940 | 100 | 75 | 5 | + 70 | 86 | .777 | + 54 | − 21 | ..... |
| ML | .00777 | .0493 | 2 @ 6 | 6550 | 100 | 51 | 5 | + 46 | 60 | .833 | + 38 | − 50 | ..... |
| NO | .01030 | .0496 | 2 @ 7 | 2307 | 100 | 24 | 5 | − 19 | 34 | .898 | − 17 | + 83 | No counter |
| AC = AD | .00390 | .0312 | 4 @ 1 | 63111 | 600 | 247 | 19 | +228 | 200 | .600 | +137 | +101 | +466 |
| BC | ..... | ..... | ..... | ..... | ... | ... | .. | −362 | ... | .... | −217 | −160 | −739 |
| AF | .00695 | .0278 | 7 @ 2 | 59095 | 2694 | 410 | 75 | +335 | 193 | .608 | +203 | +154 | +692 |
| BE | ..... | ..... | ..... | ..... | ... | ... | .. | −339 | ... | .... | −206 | −156 | −701 |
| AH | .00985 | .0263 | 11 @ 3 | 59661 | 7310 | 587 | 192 | +395 | 194 | .607 | +239 | +181 | +815 |
| BG | ..... | ..... | ..... | ..... | ... | ... | ... | −396 | ... | ... | +240 | −181 | −817 |
| BI | .01315 | .0263 | 13 @ 4 | 50901 | 9585 | 670 | 252 | −418 | 178 | .627 | −262 | −194 | −874 |
| CD | .0385 | .0770 | 4 @ 1 | 3725 | 600 | 144 | 46 | + 98 | 44 | .872 | + 86 | + 25 | +209 |

| Post at | Mem. | $M_4$ | Mc | S | $\frac{2}{3}$ D | K | $M_0$ | T | $L_1$ | $\frac{300}{L_1+300}$ | I | D.L. | Total |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 5 | JK | 22261 | 2390 | +16 | −14 | .00203 | 11340 | +23 | 114 | .725 | +17 | +3 | + 43 |
| 6 | ML | 8865 | 687 | +35 | −34 | .00214 | 5960 | +13 | 71 | .8 | +10 | +1 | + 24 |

9. Find positions of loading for maximum counter-tensions in posts and compute values by use of formulas (25) and (26).

## Problem 1.

*Calculation of Live-load Stresses in a Pratt Truss with Inclined Chord.*

The complete data for this problem are given in Fig. 13. Items 1 to 5 of the above method of procedure need no explanation. The values of the coefficients $G$ and $H$, the position of the loading for maximum stress, and the value of the maximum stress will be determined for several typical members; for example, vertical post, inclined web members, horizontal chords, end post, and inclined chords.

*Vertical Post EF.*

Formula
$$S_4 = \left(\frac{a}{bL}\right) M_4 - \left(\frac{1}{p}\right) M_3 \quad \ldots \ldots \quad (23)$$

Refer to Fig. 11 for definition of dimensions.

$$G = \frac{a}{bL} = \frac{28}{36}\,(.00480) = .00373$$

$$H = \frac{1}{p} = .0385$$

Try $w_3$ at panel point 3. Use Table 2. $L_1 = 143'$.

$$\left(\frac{G}{H} W_4 - W_3\right) = \frac{.00373}{.03850}\,(440.0) - \begin{matrix}37.5\\ \text{or}\\ 62.5\end{matrix} = \begin{matrix}+\\ \text{or}\\ -\end{matrix}$$

Therefore $w_3$ at 3 gives a maximum.

$$S = GM_4 - HM_3 = .00373(33970) - .0385(287.5)$$
$$= 126.7 - 11.0 = 115.7^k$$

$$\text{Impact factor} = \frac{300}{L_1 + 300} = \frac{300}{443} = .677$$

$$\text{Impact stress} = .677 \times 115.7 = 78.3^k.$$

*Inclined Web Member ED.*

Formula $$S_1 = \left(\frac{ta}{cbL}\right) M_4 - \left(\frac{t}{bp}\right) M_3 \quad . \; . \; . \; . \quad (24)$$

Refer to Fig. 11 for definition of dimensions.

$$G = \frac{ta}{cbL} = \frac{41.23 \times 28}{32 \times 36} (.00480) = .00481$$

$$H = \frac{t}{bp} = \frac{41.23}{36} (.0385) = .0442$$

Try $w_3$ at panel point 2. Use Table 2. $L_1 = 169'$.

$$\left(\frac{G}{H} W_4 - W_3\right) = \frac{.00481}{.0442}(505.0) - \begin{matrix} 37.5 \\ \text{or} \\ 62.5 \end{matrix} = \begin{matrix} + \\ \text{or} \\ - \end{matrix}$$

Therefore $w_3$ at 2 gives a maximum.

$$S = GM_4 - HM_3 = .00481(46255) - .0442(287.5)$$
$$= 223 - 13 = 210^k.$$

$$\text{Impact factor} = \frac{300}{469} = .640$$

$$\text{Impact stress} = .640 \times 210 = 134^k.$$

*Inclined Web Member ML.*

Formula $$S_2 = \left(\frac{ta}{cbL}\right) M_4 - \left(\frac{t}{bp}\right) M_3 \quad . \; . \; . \; . \quad (24)$$

Refer to Fig. 9 or Fig. 11 for definition of dimensions.

$$G = \frac{ta}{cbL} = \frac{46.04 \times 48}{38 \times 36} (.00480) = .00777$$

$$H = \frac{t}{bp} = \frac{46.04}{36} (.0385) = .0493$$

Try $w_2$ at panel point 6. Use Table 2. $L_1 = 60'$.

$$\left(\frac{G}{H} W_4 - W_3\right) = \frac{.00777}{.0493}(190) - \begin{matrix} 12.5 \\ \text{or} \\ 37.5 \end{matrix} = \begin{matrix} + \\ \text{or} \\ - \end{matrix}$$

Therefore $w_2$ at 6 gives a maximum.

$$S = GM_4 - HM_3 = .00777(6550) - .0493(100)$$
$$= 51 - 5 = 46^k.$$

$$\text{Impact factor} = \frac{300}{360} = .833$$

$$\text{Impact stress} = .833 \times 46 = 38^k.$$

*Lower Chord Member AC = AD.*

Formula $$S_5 = \left(\frac{m}{nv}\right)M_4 - \left(\frac{1}{v}\right)M_3 \quad . \quad . \quad . \quad . \quad . \quad (21)$$

Refer to Fig. 11 for definition of dimensions.

$$G = \frac{m}{nv} = \tfrac{1}{8}(.03125) = .00390$$

$$H = \frac{1}{v} = .0312$$

Try $w_4$ at panel point 1. . Use Table 2. $L_1 = 200'$.

$$\left(\frac{G}{H} W_4 - W_3\right) = \frac{.00390}{.0312}(582.5) - \begin{matrix} 62.5 \\ \text{or} \\ 87.5 \end{matrix} = \begin{matrix} + \\ \text{or} \\ - \end{matrix}$$

Therefore $w_4$ at 1 gives a maximum.

$$S = GM_4 - HM_3 = .00390(63111) - .0312(600)$$
$$= 247 - 19 = 228^k.$$

$$\text{Impact factor} = \frac{300}{500} = .600$$

$$\text{Impact stress} = .600 \times 228 = 137^k.$$

*End of Post BC.*

Formula $$S_6 = \frac{i}{p} S_5 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (22)$$

$$S_6 = \frac{41.23}{26}(228) = 362^k, \text{ and impact} = \frac{41.23}{26}(137) = 217^k.$$

*Lower Chord Member AH.*

Formula $$S_5 = \left(\frac{m}{nv}\right) M_4 - \left(\frac{1}{v}\right) M_3 \quad . \quad . \quad . \quad . \quad (21)$$

Refer to Fig. 11 for definition of dimensions.

$$G = \frac{m}{nv} = \tfrac{3}{8}\,(.02632) = .00985$$

$$H = \frac{1}{v} = .0263$$

Try $w_{11}$ at panel point 3. Use Table 2. $L_1 = 194'$.

$$\left(\frac{G}{H} W_4 - W_3\right) = \frac{.00985}{.0263}\,(567.5) - \begin{matrix}190\\ \text{or}\\ 215\end{matrix} = \begin{matrix}+\\ \text{or}\\ -\end{matrix}$$

Therefore $w_{11}$ at 3 gives a maximum.

$$S = GM_4 - HM_3 = .00985(59661) - .0263(7310)$$

$$= 587 - 192 = 395^k.$$

$$\text{Impact stress} = \frac{300}{494}\,S = .607 \times 395 = 239^k.$$

*Top Chord Member BG.*

Formula $$S_6 = \frac{i}{p}\,S_5 \quad \ldots\ldots\ldots\ldots \quad (22)$$

$$S_6 = \frac{26.08}{26}\,(395) = 396^k.$$

$$\text{Impact} = \frac{26.08}{26}\,(239) = 240^k.$$

*Counter-Tension in Post at Panel Point* 5.

Formulas

$$S_2 = \text{Stress } JK = \left(\frac{ta}{cbL}\right) M_4 - \left(\frac{t}{bp}\right)\left(M_3 - \frac{b}{c} M_2\right)$$

$$= \left(\frac{ta}{cbL}\right) M_4 - \left(\frac{t}{bp}\right) M_c \quad \ldots\ldots \quad (25)$$

$T$ = tension in post.

$$= \left(\frac{d_2 - d_1}{bp}\right)\left(\frac{m}{n} M_4 - M_3\right) = K \cdot M_0 \quad (26)$$

Refer to Fig. 10 for definition of dimensions.

The calculation of the dead-load compression in $JK$ is not given, but the value is $21^k$. Two-thirds of this compression, or $14^k$, will be considered effective in counterbalancing the live-load tension in $JK$. The live load must be advanced beyond the position of maximum live-load tension in $JK$ (*i.e.*, $w_2$ at panel point 5) until $S_2$, or the stress in $JK$, equals $14^k$. This must be done by trial, $S_2$ being figured each time by formula (25). It is found that when 114′ of loading has advanced upon the bridge, this condition is approximately satisfied. For this position of loading

$$M_4 = 22261$$

$$M_c = \left(M_3 - \frac{b}{c}M_2\right) = (2565 - 175) = 2390$$

$$G = \left(\frac{ta}{cbL}\right) = \frac{46.04 \times 38}{38 \times 38}(.00480) = .00580$$

$$H = \left(\frac{t}{bp}\right) = \frac{46.04}{38}(.0385) = .0466$$

Therefore,

$$S_2 = .00580(22261) - .0466(2390) = 16^k.$$

This value of $S_2 = 16^k$ balances $\frac{2}{3}\ D = -\ 14^k$, nearly enough for practical purposes. Therefore, compute $T$ for this position of the live load.

$$T = \left(\frac{d_2 - d_1}{bp}\right)\left(\frac{m}{n}M_4 - M_3\right) = K \cdot M_o$$

$$K = \frac{2 - 0}{38 \times 26} = .00203$$

$$M_o = \tfrac{5}{8}\,(22261) - 2565 = 11340$$

$$T = .00203(11340) = 23^k$$

$$\text{Impact factor} = \frac{300}{414} = .725$$

$$\text{Impact stress for } T = .725 \times 23 = 17^k.$$

## Problem 2.

*Live-load Stresses in a Pratt Truss with Parallel Chords.*

The complete data for this problem are given in Fig. 14. Formulas (21), (29), and (30) give the values of the coefficients $G$ and $H$, which are identical for several members of any Pratt truss with parallel chords. The procedure for finding the positions of the loading and maximum stresses is exactly as in Problem 1. It should be noted that

$$\text{Stress } FG = \text{Stress } EF \times \frac{37.54}{28}$$

$$\text{“} \quad HI = \quad \text{“} \quad GH \times \frac{37.54}{28}$$

$$\text{“} \quad BC = \quad \text{“} \quad AC \times \frac{37.54}{25}$$

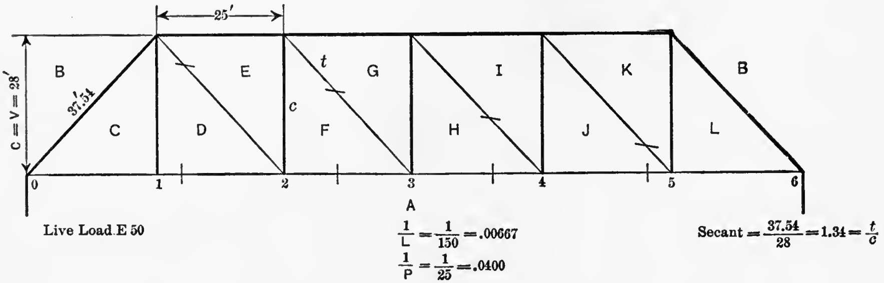


Fig. 14.

| Mem. | G | H | Wheel | $M_4$ | $M_3$ | S |
|---|---|---|---|---|---|---|
| CD | .0400 | .0800 | 4 @ 1 | 3564 | 600 | 95 |
| EF | .00667 | .0400 | 3 “ 3 | 13520 | 287 | 79 |
| FG | ...... | ..... | ...... | ..... | .... | 106 |
| GH | .00667 | .0400 | 2 “ 4 | 6170 | 100 | 37 |
| HI | ...... | ..... | ...... | ..... | .... | 50 |
| JK | .00894 | .0536 | 2 “ 5 | 2179 | 100 | 14 |
| DE | .00894 | .0536 | 3 “ 2 | 21895 | 287 | 181 |
| BC | ...... | ..... | ...... | ..... | .... | 272 |
| AC = AD | .00595 | .0357 | 4 “ 1 | 33970 | 600 | 181 |
| AF = BE | .01190 | .0357 | 7 “ 2 | 31375 | 2694 | 278 |
| BG | .01785 | .0357 | 12 “ 3 | 34411 | 8385 | 314 |

The stresses in all of the chord members may be checked by use of Table 8, and the stresses in the end post and web members may be checked by Table 9. The stress in $CD$ agrees with the maximum pier reaction in Table 7. Table 3 may be used to find the position of loading for maximum chord stresses, and Table 6 gives position of loading for maximum web stresses.

# ARTICLE VIII.

## THREE-HINGED ARCH. APPLICATION OF THE GENERAL METHOD TO THE CALCULATION OF LIVE-LOAD STRESSES.

THE general formulas $\frac{dS}{dx} = \Sigma CW$ and $S = \Sigma CM$ may be used directly to find the position of loading and the

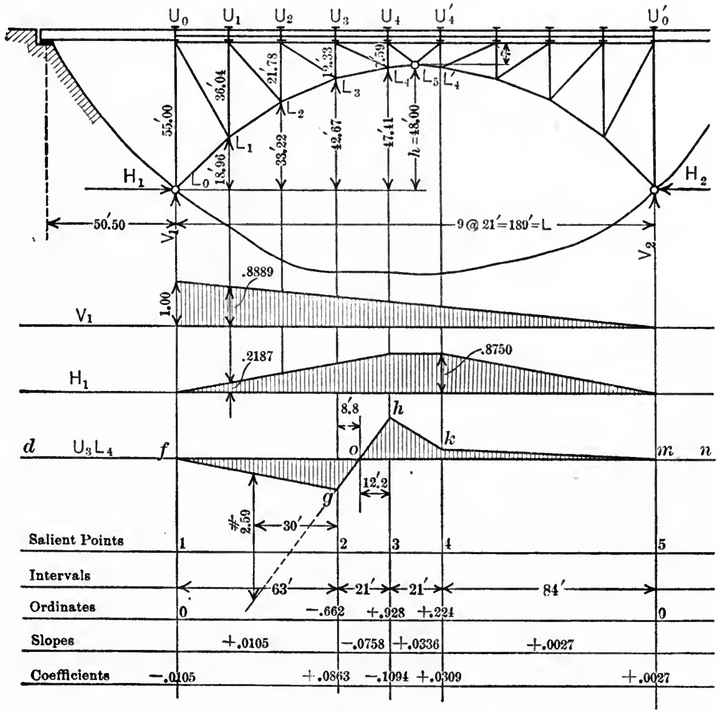


FIG 15.

value of the maximum live-load stress in any member of a framed structure as soon as the influence line for this member and the ordinates at all salient points have been determined. This method is applied to the calculation of maximum live-load stresses for the three-hinged arch shown in Fig. 15. Cooper's $E40$ loading is used.

First are drawn the influence lines for the horizontal and vertical components of the reaction at the left hinge. The vertical component $V_1$ is the same as for a simple span $L$. The horizontal component $H_1$ equals the bending moment at the centre of the span $L$ divided by the depth $h$. The influence-line ordinates for all members are now found by drawing five Maxwell diagrams, one of which is reproduced in Fig. 16. From the influence lines for $V_1$ and $H_1$, the value of $V_1$ is .8889 and $H_1$ is .2187 for a one-pound load at $U_1$. The external loads acting on the left half of the arch are then as shown in Fig. 16a. The load line *axbcya* in Fig. 16b is drawn to a scale of 10″ = 1 pound, and the Maxwell diagram completed in the usual way. The scaled

TABLE A

INFLUENCE-LINE ORDINATES FOR THREE-HINGED ARCH

| Members | Ordinates | | | | |
|---|---|---|---|---|---|
| | 1 lb. at $U_1$ | 1 lb. at $U_2$ | 1 lb. at $U_3$ | 1 lb. at $U_4$ | 1 lb. at $U'_4$ |
| $U_0U_1$ = ......... | − .403 | − .223 | − .045 | + .130 | + .201 |
| $U_1U_2$ = ......... | − .417 | − .833 | − .286 | + .262 | + .477 |
| $U_2U_3$ = ......... | − .378 | − .756 | −1.135 | + .189 | + .757 |
| $U_3U_4$ = ......... | − .171 | − .342 | − .513 | − .685 | + .548 |
| $L_0L_1$ = ......... | − .295 | − .590 | − .885 | −1.180 | −1.182 |
| $L_1L_2$ = ......... | + .221 | − .264 | − .740 | −1.224 | −1.302 |
| $L_2L_3$ = ......... | + .217 | + .434 | − .408 | −1.248 | −1.484 |
| $L_3L_4$ = ......... | + .164 | + .328 | + .491 | −1.086 | −1.674 |
| $L_4L_5$ = ......... | − .048 | − .096 | − .145 | − .193 | −1.420 |
| $U_0L_0$ = ......... | − .692 | − .384 | − .075 | + .234 | + .345 |
| $U_1L_1$ = ......... | −1.014 | − .632 | − .253 | + .129 | + .287 |
| $U_2L_2$ = ......... | + .022 | − .955 | − .490 | − .043 | + .165 |
| $U_3L_3$ = ......... | + .075 | + .150 | − .775 | − .317 | − .076 |
| $U_4L_4$ = ......... | + .114 | + .226 | + .342 | − .545 | − .364 |
| $U_0L_1$ = ......... | + .800 | + .441 | + .085 | − .270 | − .400 |
| $U_1L_2$ = ......... | + .019 | + .878 | + .350 | − .180 | − .398 |
| $U_2L_3$ = ......... | − .044 | − .088 | + .986 | + .086 | − .324 |
| $U_3L_4$ = ......... | − .221 | − .442 | − .662 | + .928 | + .224 |
| $U_4L_5$ = ......... | − .206 | − .412 | − .617 | − .823 | + .657 |
| $H$ ............. | 0.2187 | 0.4375 | 0.6562 | 0.8750 | 0.8750 |
| $V$ ............. | 0.8889 | 0.7777 | 0.6666 | 0.5555 | 0.4444 |
| $\theta$ ............. | 14° | 29° | 44° | 58° | 63° |

values of these stresses are the influence ordinates for a one pound load at $U_1$. In an exactly similar way the influence ordinates for a unit load at $U_2$, $U_3$, $U_4$, and $U'_4$ are determined. The influence lines are straight from $U'_0$ to

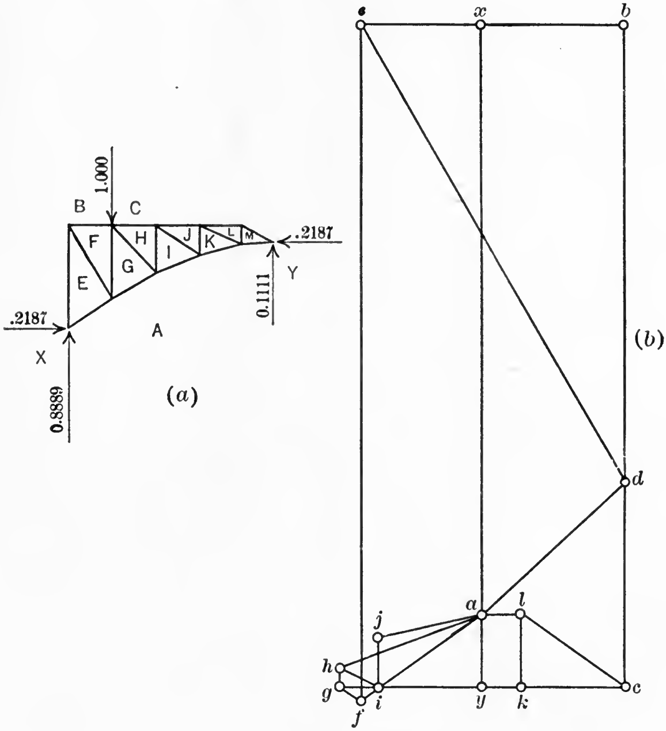


Fig 16.

$U'_4$. Table A gives the influence ordinates for all members and also for the horizontal and vertical components of the reaction at the left hinge. The angle $\theta$ is the inclination of this reaction with the vertical.

The calculation of the live-load stresses in any one member is typical. The member $U_3L_4$ is taken. The influence line for this member is drawn to scale in Fig. 15 by use of the influence ordinates from Table A. The salient points occur below panel points $U_3$, $U_4$, and $U'_4$. The distance from $U_3$ to the neutral point 0 equals $\frac{.662}{.662 + .928}(21) = 8'.8$.

*Calculation of Slopes.*

$$\text{Slope of } df = 0$$

$$fg = \frac{0 - (-.662)}{63} = +.0105$$

$$gh = \frac{-.662 - (.928)}{21} = -.0758$$

$$hk = \frac{.928 - (.224)}{21} = +.0336$$

$$km = \frac{.224 - 0}{84} = +.0027$$

$$mn = 0$$

*Calculation of Coefficients.*

$$\begin{aligned}
C_1 &= \phantom{-}0 - (.0105) &&= -.0105\\
C_2 &= \phantom{-}.0105 - (-.0758) &&= +.0863\\
C_3 &= -.0758 - (.0336) &&= -.1094\\
C_4 &= \phantom{-}.0336 - (.0027) &&= +.0309\\
C_5 &= \phantom{-}.0027 - 0 &&= +.0027
\end{aligned}$$

The sum of these coefficients equals zero. This agrees with formula (6) of Art. 3.

It should be remembered, as is pointed out in Art. 3, that the value of these coefficients may be measured graphically. For example, in Fig. 15 the value of $C_2$ is $\frac{2.59}{30} =$ .0863.

By use of the formula $\frac{dS}{dx} = \Sigma CW$ and Rule 1 of Art. 3, the position of loading for maximum tension in $U_3L_4$ may now be determined. Try wheel 3 at $U_4$ with the loading advancing toward the left. Take the values of the load sums and moment sums for $E40$ from Table 2.

$$\frac{dS}{dx} = \Sigma CW = -.1094(30) + .309(103) + .0027(302) = +.7$$

$$\frac{dS}{dx} = \Sigma CW = -.1094(50) + .309(103) + .0027(302) = -.7$$

Therefore $w_3$ at $U_4$ gives a maximum tension in $U_3L_4$, and its value is

$$S = \Sigma CM = -.1094(230) + .309(1846) + .0027(19001) = 83^k.$$

By use of the formula $\frac{dS}{dx} = \Sigma CW$ and Rule 2 of Art. 3, the position of loading for maximum compression in $U_3L_4$ is now determined. Try wheel 2 at $U_3$ with the loading advancing toward the right. Note that the signs of the coefficients remain unchanged. Take the values of the load sums and moment sums for $E40$ from Table 2.

$$\frac{dS}{dx} = \Sigma CW = -.0105(192) + .0863(10) = -1.3$$

$$\frac{dS}{dx} = \Sigma CW = -.0105(192) + .0863(30) = +0.6$$

Therefore $w_2$ at $U_3$ gives a maximum negative stress, or compression, in $U_3L_4$, and its value is

$$S = \Sigma CM = -.0105(7092) + .0863(80) = -67^k.$$

The above values of $83^k$ and $67^k$ for maximum tension and compression in $U_3L_4$ may be checked by use of formula $S = qA_z$ (2), the values of $q$ being taken from Table 16.

*Tension $U_3L_4$ by Equivalent Uniform Load.*

The area of the tension part of the influence line equals

$$A_z = 27.2$$

The influence line *ohkm* is not triangular, but a triangular influence line with intervals $l_1 = 10$ ft. and $l_2 = 45$ ft. approximates its shape closely enough for the selection of an equivalent uniform load. For $l_1 = 10'$ and $l_2 = 45'$, Table 16 gives $3.080^k$ as the equivalent uniform load.

Therefore,

$$S = qA_z = (3.080)\ (27.2) = 84^k.$$

This value checks very closely that obtained by the exact method.

*Compression $U_3L_4$ by Equivalent Uniform Load.*

Choose from Table 16 the equivalent uniform load for $l_1 = 10$ ft. and $l_2 = 65$ ft. From the influence line $A_z = 23.7$.

Therefore,

$$S = qA_z = (2.870)\ (23.7) = 68^k.$$

This checks closely the value obtained by the exact method.

Calculation of other members of this arch and of some more complicated framed structures shows a close agreement between the two preceding methods. The latter method is the simpler when a table of equivalent uniform loads has been made, especially in the case of the more complex influence lines for members of swing bridges, two-hinged arches, arch ribs, etc. The method of calculating a table of equivalent uniform loads will be explained in the following article.

# ARTICLE IX.

## EQUIVALENT UNIFORM LOADS.

An equivalent uniform load is one which gives the same stress as does a loading which is not uniform. For any given standard loading, the equivalent uniform load is different for stresses whose influence lines differ. Since the forms of influence lines are innumerable, a table of exact equivalent uniform loads for all stresses is impracticable. A table of equivalent uniform loads, however, for stresses whose influence lines are *triangular* may be used with little error in selecting equivalent uniform loads for stresses whose influence lines are *not triangular*. It is, therefore, sufficient for practical purposes to make tables of equivalent uniform loads for a series of *triangular* influence lines. It may be shown that the equivalent uniform load for any triangular influence line is dependent entirely upon the intervals $l_1$ and $l_2$, and is independent of the ordinate $h$ at the apex of the influence line. Consider the triangular influence line in Fig. 1b to be for any stress $S$. Let the ordinate below $C$ be any value $h$. If $q$ equals the equivalent uniform load covering $l_1$ and $l_2$,

$$S = qA_z, \text{ or } q = \frac{S}{A_z} \quad . \ . \ . \ . \ . \ . \ . \quad (A)$$

The area of this influence line is

$$A_z = \frac{h}{2}(l_1 + l_2) = \frac{h}{2} L \quad . \ . \ . \ . \ . \ . \ . \quad (B)$$

Furthermore, if the concentrated live loads have been placed so as to give the maximum pier reaction between two spans $l_1$ and $l_2$, this same position of loading will give maximum $S$, if the influence line for $S$ is a triangle with the same intervals $l_1$ and $l_2$. Since the influence ordinates for $S$ are related to the influence ordinates for $R$ as $h$ is to unity,

$$\frac{S}{R} = \frac{h}{1.00}$$

Or

$$S = hR \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (C)$$

Substituting the values of $A_z$ and $S$ from equations $(B)$ and $(C)$ in equation $(A)$,

$$q = hR \div \frac{h}{2} L = \frac{2R}{L} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (D)$$

It appears, therefore, that $q$ is independent of $h$.
From formula (16) of Art. 5,

$$R = \frac{L}{l_1 l_2} M \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . (16)$$

Substituting for $R$ in equation $(D)$,

$$q = \frac{2R}{L} = \frac{2M}{l_1 l_2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (31)$$

The term $M$ is the bending moment in the span $L = l_1 + l_2$ at the point where the intervals are $l_1$ and $l_2$.

Tables (10) to (18) inclusive have been calculated for the positions of the live load given by Table 3. The values of $M$ were first found, then the values of $R$, and finally the values of the equivalent uniform loads. The three formulas that were used in succession are

$$M = \frac{l_1}{L} M_3 + \frac{l_2}{L} M_1 - M_2 \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

$$R = \frac{L}{l_1 l_2} M \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (16)$$

$$q = \frac{2M}{l_1 l_2} = \frac{2R}{L} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (31)$$

An example of the use of equivalent uniform loads has already been given in Art. 8. The general formula $S = qA_z$ may be used in any case. For the special cases of bending moment in a beam and pier reaction between two simple spans, formula (31) gives

$$M = q\left(\frac{l_1 l_2}{2}\right) \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (32)$$

$$R = q\left(\frac{L}{2}\right) = q\left(\frac{l_1 + l_2}{2}\right) . \; . \; . \; . \; . \; . \quad (33)$$

The quantities in the parentheses are the areas of the influence lines for $M$ and $R$ respectively.

# ARTICLE X.

## METHOD OF CALCULATING TABLE OF LOAD SUMS FOR ANY STANDARD LOADING. ILLUSTRATIVE EXAMPLE.

THE definitions of *moment sum* and *load sum* are given at the beginning of Art. 2. It is at once evident that a table of *load sums* may be computed by adding the successive loads. It may be shown that the table of moment sums may also be calculated by the process of addition.

From formula (5a) of Art. 2,

$$C_a W_a = C_a \frac{dM_a}{dx}$$

Or

$$dM_a = W_a \cdot dx.$$

Expressed in words, the increase in the *moment sum* for an increase $dx$ in the distance of the centre of moments from wheel 1 equals the *load sum* times $dx$. If the load sum is constant for an interval $dx = 1$ foot, as between concentrated loads, the increase of the moment sum for $dx =$ 1 foot equals the corresponding load sum. If the load sum is not constant, but *uniformly* increasing, as when the centre of moments lies within the uniform load, the increase of the moment sum for $dx = 1$ foot equals the *average* value of the load sum for this one foot interval. The application of the foregoing principles is made clear by the following example.

*Example.*—Give explicit directions for the calculation of a table of load sums and moment sums at intervals of 1 foot from 0′ to 400′ for Cooper's $E40$ loading.

*Solution.*—Calculate the table of load sums by adding the loads one by one, taking a sub-total for each addition. Thus, the following numbers are added:

1—10
4—20's
4—13's
1—10
4—20's
4—13's
391— 2's

If the final total checks $284 + 391 \times 2 = 866$, the table of load sums is correct.

Assume now that the table of load sums for *E*40 has been completed. The table of moment sums may now be found as directed below. The following numbers are to be added one by one, taking a sub-total for each addition:

8—10's
5—30's
5—50's
5—70's
9—90's
5—103's
6—116's
5—129's
8—142's
8—152's
5—172's
5—192's
5—212's
9—232's
5—245's
6—258's
5—271's
5—284's
1—285
1—287
1—289

and all odd numbers up to 865.

If the final total checks up 183,689, which is figured independently, the table of moment sums is correct.

The preceding additions may be made most satisfactorily on a recording adding machine. Table 2 was calculated in this way.

It will be noted that the table of load sums serves as a table of differences for the table of moment sums.

# ARTICLE XI.

## SUMMARY OF FORMULAS.

*Art.* 1.

$$Z = \Sigma wz \quad \ldots \quad (1)$$

$$Z = qA_z \quad \ldots \quad (2)$$

$$Z = w\Sigma z \quad \ldots \quad (3)$$

$$Z = z\Sigma w = zW \quad \ldots \quad (4)$$

*Art.* 2.

$$Z = \Sigma w_n z_n = C_a \Sigma w_n x_n = C_a M_a \quad \ldots \quad (5)$$

$$\frac{dZ}{dx} = C_a W_a = \frac{d\,(C_a M_a)}{dx} = \frac{C_a dM_a}{dx} \quad \ldots \quad (5a)$$

*Art.* 3.

$$\Sigma C = 0 \quad \ldots \quad (6)$$

$$S = \Sigma CM \quad \ldots \quad (7)$$

$$\frac{dS}{dx} = \Sigma CW \quad \ldots \quad (8)$$

*Art.* 4. *Girder Bridge without Panels.*

End reactions.

$$R_1 = \frac{M_3 - M_1}{L} - W_1 \quad \ldots \quad (9)$$

$$R_2 = W_3 - \frac{M_3 - M_1}{L} \quad \ldots \quad (9a)$$

Bending moment for unequal segments $l_1$ and $l_2$.

$$M = \frac{l_1}{L} M_3 + \frac{l_2}{L} M_1 - M_2 \quad \ldots \quad (10)$$

$$\frac{dM}{dx} = \frac{l_1}{L} W_3 + \frac{l_2}{L} W_1 - W_2 \quad \ldots \quad (11)$$

Bending moment at centre. $l_1 = l_2 = \frac{L}{2}$

$$M = \frac{M_3 + M_1}{2} - M_2 \quad \text{(10a)}$$

$$\frac{dM}{dx} = \frac{W_3 + W_1}{2} - W_2 \quad \text{(11a)}$$

Shear at any section.

$$S = \frac{M_3 - M_1}{L} - W_2 \quad \text{(12)}$$

Location of centre of gravity of loading on span.

$$\bar{x} = \frac{M_3 - M_1 - LW_1}{W_3 - W_1} \quad \text{(13)}$$

When $M_1 = 0$,

$$\bar{x} = \frac{M_3}{W_3} \quad \text{(13a)}$$

*Art. 5. Pier Reaction.*

For unequal spans $l_1$ and $l_2$.

$$R = \frac{M_3}{l_2} + \frac{M_1}{l_1} - \frac{L}{l_1 l_2}M_2 = \frac{L}{l_1 l_2}\left(\frac{l_1}{L}M_3 + \frac{l_2}{L}M_1 - M_2\right) \quad \text{(14)}$$

$$\frac{dR}{dx} = \frac{W_3}{l_2} + \frac{W_1}{l_1} - \frac{L}{l_1 l_2}W_2 = \frac{L}{l_1 l_2}\left(\frac{l_1}{L}W_3 + \frac{l_2}{L}W_1 - W_2\right) \quad \text{(15)}$$

For equal spans $l_1$ and $l_2$ equal to $l$.

$$R = \frac{M_3 + M_1 - 2M_2}{l} \quad \text{(14a)}$$

$$\frac{dR}{dx} = \frac{W_3 + W_1 - 2W_2}{l} \quad \text{(15a)}$$

Relation between $R$ and $M$,

$$R = \frac{L}{l_1 l_2}M \quad \text{(16)}$$

*Art. 6. Girder Bridge with Panels.*

Shear in end panel; general case.

$$S_a = \frac{1}{L}M_3 + \frac{l_2}{l_1 L}M_1 - \frac{1}{l_1}M_2 = \frac{1}{l_1}\left(\frac{l_1}{L}M_3 + \frac{l_2}{L}M_1 - M_2\right) \quad \text{(17)}$$

$$\frac{dS_a}{dx} = \frac{1}{L}W_3 + \frac{l_2}{l_1 L}W_1 - \frac{1}{l_1}W_2 = \frac{1}{l_1}\left(\frac{l_1}{L}W_3 + \frac{l_2}{L}W_1 - W_2\right) \quad (18)$$

Shear in intermediate panel; general case.

$$S_b = \frac{M_4}{L} - \frac{M_3}{p} + \frac{M_2}{p} - \frac{M_1}{L} \quad . \quad . \quad . \quad . \quad (19)$$

$$\frac{dS_b}{dx} = \frac{W_4}{L} - \frac{W_3}{p} + \frac{W_2}{p} - \frac{W_1}{L} \quad . \quad . \quad . \quad . \quad . \quad . \quad (20)$$

Shear in intermediate panel; usual case.

$$S = \frac{M_4}{L} - \frac{M_3}{p} = \frac{1}{p}\left(\frac{p}{L}M_4 - M_3\right) \quad . \quad (19a)$$

$$\frac{dS_b}{dx} = \frac{W_4}{L} - \frac{W_3}{p} = \frac{1}{p}\left(\frac{p}{L}W_4 - W_3\right) \quad . \quad (20a)$$

*Art. 7. Through Pratt Truss with Inclined Chord.*

Stress in hanger. Use formulas (14a) and (15a).
Stress in any horizontal chord member; usual case.

$$S_5 = \left(\frac{m}{nv}\right)M_4 - \left(\frac{1}{v}\right)M_3 \quad . \quad . \quad . \quad . \quad (21)$$

Compression in any inclined top chord member or end post; usual case.

$$S_6 = \left(\frac{i}{p}\right)S_5 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (22)$$

Compression in vertical post; usual case.

$$S_4 = \left(\frac{a}{bL}\right) M_4 - \left(\frac{1}{p}\right) M_3 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (23)$$

Stresses in inclined web members including counters; usual case.

$$S_1, S_2, S_3 = \left(\frac{ta}{cbL}\right)M_4 - \left(\frac{t}{bp}\right)M_3 \quad . \quad . \quad . \quad (24)$$

Stress in inclined counter; special case of loading advanced beyond panel.

$$S_2 = \left(\frac{ta}{cbL}\right)M_4 - \frac{t}{bp}\left(M_3 - \frac{b}{c}M_2\right) = \left(\frac{ta}{cbL}\right)M_4 - \left(\frac{t}{bp}\right)M_c \quad (25)$$

Counter-tension in vertical post; usual case.

$$T = \left(\frac{d_2 - d_1}{bp}\right)\left(\frac{m}{n}M_4 - M_3\right) = K \cdot M_o \quad . \; . \; (26)$$

Formulas (21), (23), and (24) are of the general form

$$S = GM_4 - HM_3 \quad . \; . \; . \; . \; . \; . \; (27)$$

where the coefficients $G$ and $H$ may be tabulated thus:

| *Type of member* | $G$ | $H$ |
|---|---|---|
| Horizontal chord | $\frac{m}{nv}$ | $\frac{1}{v}$ |
| Vertical post | $\frac{a}{bL}$ | $\frac{1}{p}$ |
| Inclined web member | $\frac{ta}{cbL}$ | $\frac{t}{bp}$ |

The rate of variation of $S$ in formula (27) is

$$\frac{dS}{dx} = GW_4 - HW_3 = H\left(\frac{G}{H}W_4 - W_3\right) \quad . \; . \; (28)$$

When $S$ in formulas (21), (23), or (24) is a maximum $\left(\frac{G}{H}W_4 - W_3\right)$ passes through zero.

*Through Pratt Truss—Parallel Chords.*

Stress in hanger,—use formulas (14a) and (15a)

$$\text{Stress in horizontal chord} = S_5 = \left(\frac{m}{nv}\right)M_4 - \left(\frac{1}{v}\right)M_3 \quad . \; (21)$$

$$\text{" \quad " vertical post} = S_4 = \left(\frac{1}{L}\right)M_4 - \left(\frac{1}{p}\right)M_3 \quad . \; . \; (29)$$

$$\text{" \quad " inclined web} = S_1 = \left(\frac{t}{cL}\right)M_4 - \left(\frac{t}{cp}\right)M_3 = \frac{t}{c}S_4 \quad (30)$$

Stress in end post $= S_6 = \frac{\;}{p} S_5 \quad \ldots \ldots \ldots \quad (22)$

Formulas (21), (29), and (30) are of the general form

$$S = G \cdot M_4 - H \cdot M_3 \quad \ldots \ldots \quad (27)$$

and their rate of variation is

$$\frac{dS}{dx} = H\left(\frac{G}{H} W_4 - W_3\right) \quad \ldots \ldots \quad (28)$$

$G$ and $H$ are the coefficients of $M_4$ and $M_3$ in equations (21), (29), and (30), respectively.

When $S$ in formulas (21), (29), or (30) is a maximum, $\left(\frac{G}{H} W_4 - W_3\right)$ passes through zero.

### *Art.* 9. *Equivalent Uniform Loads.*

$$q = \frac{2M}{l_1 l_2} = \frac{2R}{L} \quad \ldots \ldots \ldots \quad (31)$$

$$M = q\left(\frac{l_1 l_2}{2}\right) \quad \ldots \ldots \ldots \quad (32)$$

$$R = q\left(\frac{L}{2}\right) = q\left(\frac{l_1 + l_2}{2}\right) \quad \ldots \ldots \quad (33)$$

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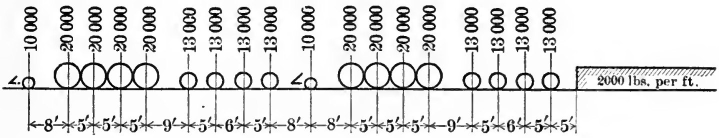
# TABLE 1

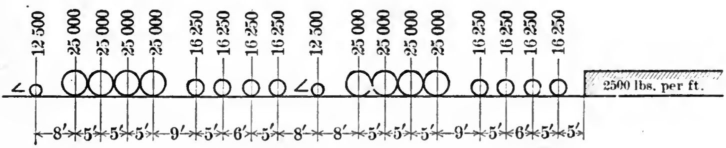
## STANDARD LOADINGS

**Loads given are for one rail.**

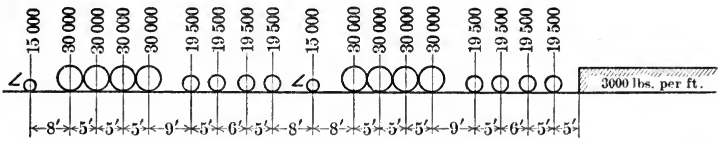
COOPER'S E 40:



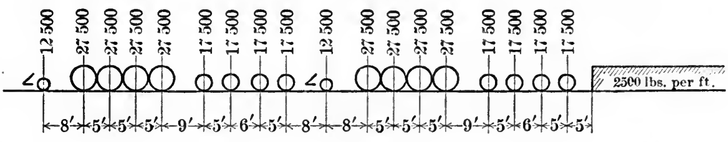

COOPER'S E 50:



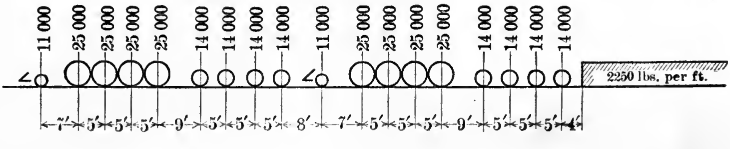

COOPER'S E 60:




COMMON STANDARD-1904-PACIFIC SYSTEM




D. L. & W. R. R.:



# TABLE 2

## LOAD SUMS AND MOMENT SUMS FOR COOPER'S AND OTHER STANDARD LOADINGS

NOTE.—Load Sums and Moment Sums are given per rail in thousands of pounds and foot-pounds respectively.

Cooper's $E40$. 0′–50′

| Length | Wheel | Load | Load Sums | Moment Sums |
|---|---|---|---|---|
| 0 | w. 1 | 10 | 10 | 0 |
| 1 | .... | .. | ... | 10 |
| 2 | .... | .. | ... | 20 |
| 3 | .... | .. | ... | 30 |
| 4 | .... | .. | ... | 40 |
| 5 | .... | .. | ... | 50 |
| 6 | .... | .. | ... | 60 |
| 7 | .... | .. | ... | 70 |
| 8 | w. 2 | 20 | 30 | 80 |
| 9 | .... | .. | ... | 110 |
| 10 | .... | .. | ... | 140 |
| 11 | .... | .. | ... | 170 |
| 12 | .... | .. | ... | 200 |
| 13 | w. 3 | 20 | 50 | 230 |
| 14 | .... | .. | ... | 280 |
| 15 | .... | .. | ... | 330 |
| 16 | .... | .. | ... | 380 |
| 17 | .... | .. | ... | 430 |
| 18 | w. 4 | 20 | 70 | 480 |
| 19 | .... | .. | ... | 550 |
| 20 | .... | .. | ... | 620 |
| 21 | .... | .. | ... | 690 |
| 22 | .... | .. | ... | 760 |
| 23 | w. 5 | 20 | 90 | 830 |
| 24 | .... | .. | ... | 920 |
| 25 | .... | .. | ... | 1010 |
| 26 | .... | .. | ... | 1100 |
| 27 | .... | .. | ... | 1190 |
| 28 | .... | .. | ... | 1280 |
| 29 | .... | .. | ... | 1370 |
| 30 | .... | .. | ... | 1460 |
| 31 | .... | .. | ... | 1550 |
| 32 | w. 6 | 13 | 103 | 1640 |
| 33 | .... | .. | ... | 1743 |
| 34 | .... | .. | ... | 1846 |
| 35 | .... | .. | ... | 1949 |
| 36 | .... | .. | ... | 2052 |
| 37 | w. 7 | 13 | 116 | 2155 |
| 38 | .... | .. | ... | 2271 |
| 39 | .... | .. | ... | 2387 |
| 40 | .... | .. | ... | 2503 |
| 41 | .... | .. | ... | 2619 |
| 42 | .... | .. | ... | 2735 |
| 43 | w. 8 | 13 | 129 | 2851 |
| 44 | .... | .. | ... | 2980 |
| 45 | .... | .. | ... | 3109 |
| 46 | .... | .. | ... | 3238 |
| 47 | .... | .. | ... | 3367 |
| 48 | w. 9 | 13 | 142 | 3496 |
| 49 | .... | .. | ... | 3638 |
| 50 | .... | .. | ... | 3780 |

Cooper's $E40$. 50′–100′

| Length | Wheel | Load | Load Sums | Moment Sums |
|---|---|---|---|---|
| 50 | ...... | .. | ... | 3780 |
| 51 | ...... | .. | ... | 3922 |
| 52 | ...... | .. | ... | 4064 |
| 53 | ...... | .. | ... | 4206 |
| 54 | ...... | .. | ... | 4348 |
| 55 | ...... | .. | ... | 4490 |
| 56 | w. 10 | 10 | 152 | 4632 |
| 57 | ...... | .. | ... | 4784 |
| 58 | ...... | .. | ... | 4936 |
| 59 | ...... | .. | ... | 5088 |
| 60 | ...... | .. | ... | 5240 |
| 61 | ...... | .. | ... | 5392 |
| 62 | ...... | .. | ... | 5544 |
| 63 | ...... | .. | ... | 5696 |
| 64 | w. 11 | 20 | 172 | 5848 |
| 65 | ...... | .. | ... | 6020 |
| 66 | ...... | .. | ... | 6192 |
| 67 | ...... | .. | ... | 6364 |
| 68 | ...... | .. | ... | 6536 |
| 69 | w. 12 | 20 | 192 | 6708 |
| 70 | ...... | .. | ... | 6900 |
| 71 | ...... | .. | ... | 7092 |
| 72 | ...... | .. | ... | 7284 |
| 73 | ...... | .. | ... | 7476 |
| 74 | w. 13 | 20 | 212 | 7668 |
| 75 | ...... | .. | ... | 7880 |
| 76 | ...... | .. | ... | 8092 |
| 77 | ...... | .. | ... | 8304 |
| 78 | ...... | .. | ... | 8516 |
| 79 | w. 14 | 20 | 232 | 8728 |
| 80 | ...... | .. | ... | 8960 |
| 81 | ...... | .. | ... | 9192 |
| 82 | ...... | .. | ... | 9424 |
| 83 | ...... | .. | ... | 9656 |
| 84 | ...... | .. | ... | 9888 |
| 85 | ...... | .. | ... | 10120 |
| 86 | ...... | .. | ... | 10352 |
| 87 | ...... | .. | ... | 10584 |
| 88 | w. 15 | 13 | 245 | 10816 |
| 89 | ...... | .. | ... | 11061 |
| 90 | ...... | .. | ... | 11306 |
| 91 | ...... | .. | ... | 11551 |
| 92 | ...... | .. | ... | 11796 |
| 93 | w. 16 | 13 | 258 | 12041 |
| 94 | ...... | .. | ... | 12299 |
| 95 | ...... | .. | ... | 12557 |
| 96 | ...... | .. | ... | 12815 |
| 97 | ...... | .. | ... | 13073 |
| 98 | ...... | .. | ... | 13331 |
| 99 | w. 17 | 13 | 271 | 13589 |
| 100 | ...... | .. | ... | 13860 |

Cooper's *E*40. 100′–150′

| Length | Wheel | Load | Load Sums | Moment Sums |
|---|---|---|---|---|
| 100 | ....... | ....... | ....... | 13860 |
| 101 | ....... | ....... | ....... | 14131 |
| 102 | ....... | ....... | ....... | 14402 |
| 103 | ....... | ....... | ....... | 14673 |
| 104 | w. 18 | 13 | 284 | 14944 |
| 105 | ....... | ....... | ....... | 15228 |
| 106 | ....... | ....... | ....... | 15512 |
| 107 | ....... | ....... | ....... | 15796 |
| 108 | ....... | ....... | ....... | 16080 |
| 109 | ....... | ....... | 284 | 16364 |
| 110 | ....... | Uniform Load = 2,000 pounds per foot | 286 | 16649 |
| 111 | ....... | | 288 | 16936 |
| 112 | ....... | | 290 | 17225 |
| 113 | ....... | | 292 | 17516 |
| 114 | ....... | | 294 | 17809 |
| 115 | ....... | | 296 | 18104 |
| 116 | ....... | | 298 | 18401 |
| 117 | ....... | | 300 | 18700 |
| 118 | ....... | | 302 | 19001 |
| 119 | ....... | | 304 | 19304 |
| 120 | ....... | | 306 | 19609 |
| 121 | ....... | | 308 | 19916 |
| 122 | ....... | | 310 | 20225 |
| 123 | ....... | | 312 | 20536 |
| 124 | ....... | | 314 | 20849 |
| 125 | ....... | | 316 | 21164 |
| 126 | ....... | | 318 | 21481 |
| 127 | ....... | | 320 | 21800 |
| 128 | ....... | | 322 | 22121 |
| 129 | ....... | | 324 | 22444 |
| 130 | ....... | | 326 | 22769 |
| 131 | ....... | | 328 | 23096 |
| 132 | ....... | | 330 | 23425 |
| 133 | ....... | | 332 | 23756 |
| 134 | ....... | | 334 | 24089 |
| 135 | ....... | | 336 | 24424 |
| 136 | ....... | | 338 | 24761 |
| 137 | ....... | | 340 | 25100 |
| 138 | ....... | | 342 | 25441 |
| 139 | ....... | | 344 | 25784 |
| 140 | ....... | | 346 | 26129 |
| 141 | ....... | | 348 | 26476 |
| 142 | ....... | | 350 | 26825 |
| 143 | ....... | | 352 | 27176 |
| 144 | ....... | | 354 | 27529 |
| 145 | ....... | | 356 | 27884 |
| 146 | ....... | | 358 | 28241 |
| 147 | ....... | | 360 | 28600 |
| 148 | ....... | | 362 | 28961 |
| 149 | ....... | | 364 | 29324 |
| 150 | ....... | | 366 | 29689 |

Cooper's *E*40. 150′–200′

| Length | Load | Load Sums | Moment Sums |
|---|---|---|---|
| 150 | Uniform Load = 2,000 pounds per foot | 366 | 29689 |
| 151 | | 368 | 30056 |
| 152 | | 370 | 30425 |
| 153 | | 372 | 30796 |
| 154 | | 374 | 31169 |
| 155 | | 376 | 31544 |
| 156 | | 378 | 31921 |
| 157 | | 380 | 32300 |
| 158 | | 382 | 32681 |
| 159 | | 384 | 33064 |
| 160 | | 386 | 33449 |
| 161 | | 388 | 33836 |
| 162 | | 390 | 34225 |
| 163 | | 392 | 34616 |
| 164 | | 394 | 35009 |
| 165 | | 396 | 35404 |
| 166 | | 398 | 35801 |
| 167 | | 400 | 36200 |
| 168 | | 402 | 36601 |
| 169 | | 404 | 37004 |
| 170 | | 406 | 37409 |
| 171 | | 408 | 37816 |
| 172 | | 410 | 38225 |
| 173 | | 412 | 38636 |
| 174 | | 414 | 39049 |
| 175 | | 416 | 39464 |
| 176 | | 418 | 39881 |
| 177 | | 420 | 40300 |
| 178 | | 422 | 40721 |
| 179 | | 424 | 41144 |
| 180 | | 426 | 41569 |
| 181 | | 428 | 41996 |
| 182 | | 430 | 42425 |
| 183 | | 432 | 42856 |
| 184 | | 434 | 43289 |
| 185 | | 436 | 43724 |
| 186 | | 438 | 44161 |
| 187 | | 440 | 44600 |
| 188 | | 442 | 45041 |
| 189 | | 444 | 45484 |
| 190 | | 446 | 45929 |
| 191 | | 448 | 46376 |
| 192 | | 450 | 46825 |
| 193 | | 452 | 47276 |
| 194 | | 454 | 47729 |
| 195 | | 456 | 48184 |
| 196 | | 458 | 48641 |
| 197 | | 460 | 49100 |
| 198 | | 462 | 49561 |
| 199 | | 464 | 50024 |
| 200 | | 466 | 50489 |

Cooper's $E$40. 200′–250′ Cooper's $E$40. 250′–300′

| Length | Load | Load Sums | Moment Sums | Length | Load | Load Sums | Moment Sums |
|---|---|---|---|---|---|---|---|
| 200 | Uniform Load = 2,000 pounds per foot | 466 | 50489 | 250 | Uniform Load = 2,000 pounds per foot | 566 | 76289 |
| 201 | | 468 | 50956 | 251 | | 568 | 76856 |
| 202 | | 470 | 51425 | 252 | | 570 | 77425 |
| 203 | | 472 | 51896 | 253 | | 572 | 77996 |
| 204 | | 474 | 52369 | 254 | | 574 | 78569 |
| 205 | | 476 | 52844 | 255 | | 576 | 79144 |
| 206 | | 478 | 53321 | 256 | | 578 | 79721 |
| 207 | | 480 | 53800 | 257 | | 580 | 80300 |
| 208 | | 482 | 54281 | 258 | | 582 | 80881 |
| 209 | | 484 | 54764 | 259 | | 584 | 81464 |
| 210 | | 486 | 55249 | 260 | | 586 | 82049 |
| 211 | | 488 | 55736 | 261 | | 588 | 82636 |
| 212 | | 490 | 56225 | 262 | | 590 | 83225 |
| 213 | | 492 | 56716 | 263 | | 592 | 83816 |
| 214 | | 494 | 57209 | 264 | | 594 | 84409 |
| 215 | | 496 | 57704 | 265 | | 596 | 85004 |
| 216 | | 498 | 58201 | 266 | | 598 | 85601 |
| 217 | | 500 | 58700 | 267 | | 600 | 86200 |
| 218 | | 502 | 59201 | 268 | | 602 | 86801 |
| 219 | | 504 | 59704 | 269 | | 604 | 87404 |
| 220 | | 506 | 60209 | 270 | | 606 | 88009 |
| 221 | | 508 | 60716 | 271 | | 608 | 88616 |
| 222 | | 510 | 61225 | 272 | | 610 | 89225 |
| 223 | | 512 | 61736 | 273 | | 612 | 89836 |
| 224 | | 514 | 62249 | 274 | | 614 | 90449 |
| 225 | | 516 | 62764 | 275 | | 616 | 91064 |
| 226 | | 518 | 63281 | 276 | | 618 | 91681 |
| 227 | | 520 | 63800 | 277 | | 620 | 92300 |
| 228 | | 522 | 64321 | 278 | | 622 | 92921 |
| 229 | | 524 | 64844 | 279 | | 624 | 93544 |
| 230 | | 526 | 65369 | 280 | | 626 | 94169 |
| 231 | | 528 | 65896 | 281 | | 628 | 94796 |
| 232 | | 530 | 66425 | 282 | | 630 | 95425 |
| 233 | | 532 | 66956 | 283 | | 632 | 96056 |
| 234 | | 534 | 67489 | 284 | | 634 | 96689 |
| 235 | | 536 | 68024 | 285 | | 636 | 97324 |
| 236 | | 538 | 68561 | 286 | | 638 | 97961 |
| 237 | | 540 | 69100 | 287 | | 640 | 98600 |
| 238 | | 542 | 69641 | 288 | | 642 | 99241 |
| 239 | | 544 | 70184 | 289 | | 644 | 99884 |
| 240 | | 546 | 70729 | 290 | | 646 | 100529 |
| 241 | | 548 | 71276 | 291 | | 648 | 101176 |
| 242 | | 550 | 71825 | 292 | | 650 | 101825 |
| 243 | | 552 | 72376 | 293 | | 652 | 102476 |
| 244 | | 554 | 72929 | 294 | | 654 | 103129 |
| 245 | | 556 | 73484 | 295 | | 656 | 103784 |
| 246 | | 558 | 74041 | 296 | | 658 | 104441 |
| 247 | | 560 | 74600 | 297 | | 660 | 105100 |
| 248 | | 562 | 75161 | 298 | | 662 | 105761 |
| 249 | | 564 | 75724 | 299 | | 664 | 106424 |
| 250 | | 566 | 76289 | 300 | | 666 | 107089 |

Cooper's $E40$. 300′–350′ | Cooper's $E40$. 350′–400′

| Length | Load | Load Sums | Moment Sums | Length | Load | Load Sums | Moment Sums |
|---|---|---|---|---|---|---|---|
| 300 | Uniform Load = 2,000 pounds per foot | 666 | 107089 | 350 | Uniform Load = 2,000 pounds per foot | 766 | 142889 |
| 301 | | 668 | 107756 | 351 | | 768 | 143656 |
| 302 | | 670 | 108425 | 352 | | 770 | 144425 |
| 303 | | 672 | 109096 | 353 | | 772 | 145196 |
| 304 | | 674 | 109769 | 354 | | 774 | 145969 |
| 305 | | 676 | 110444 | 355 | | 776 | 146744 |
| 306 | | 678 | 111121 | 356 | | 778 | 147521 |
| 307 | | 680 | 111800 | 357 | | 780 | 148300 |
| 308 | | 682 | 112481 | 358 | | 782 | 149081 |
| 309 | | 684 | 113164 | 359 | | 784 | 149864 |
| 310 | | 686 | 113849 | 360 | | 786 | 150649 |
| 311 | | 688 | 114536 | 361 | | 788 | 151436 |
| 312 | | 690 | 115225 | 362 | | 790 | 152225 |
| 313 | | 692 | 115916 | 363 | | 792 | 153016 |
| 314 | | 694 | 116609 | 364 | | 794 | 153809 |
| 315 | | 696 | 117304 | 365 | | 796 | 154604 |
| 316 | | 698 | 118001 | 366 | | 798 | 155401 |
| 317 | | 700 | 118700 | 367 | | 800 | 156200 |
| 318 | | 702 | 119401 | 368 | | 802 | 157001 |
| 319 | | 704 | 120104 | 369 | | 804 | 157804 |
| 320 | | 706 | 120809 | 370 | | 806 | 158609 |
| 321 | | 708 | 121516 | 371 | | 808 | 159416 |
| 322 | | 710 | 122225 | 372 | | 810 | 160225 |
| 323 | | 712 | 122936 | 373 | | 812 | 161036 |
| 324 | | 714 | 123649 | 374 | | 814 | 161849 |
| 325 | | 716 | 124364 | 375 | | 816 | 162664 |
| 326 | | 718 | 125081 | 376 | | 818 | 163481 |
| 327 | | 720 | 125800 | 377 | | 820 | 164300 |
| 328 | | 722 | 126521 | 378 | | 822 | 165121 |
| 329 | | 724 | 127244 | 379 | | 824 | 165944 |
| 330 | | 726 | 127969 | 380 | | 826 | 166769 |
| 331 | | 728 | 128696 | 381 | | 828 | 167596 |
| 332 | | 730 | 129425 | 382 | | 830 | 168425 |
| 333 | | 732 | 130156 | 383 | | 832 | 169256 |
| 334 | | 734 | 130889 | 384 | | 834 | 170089 |
| 335 | | 736 | 131624 | 385 | | 836 | 170924 |
| 336 | | 738 | 132361 | 386 | | 838 | 171761 |
| 337 | | 740 | 133100 | 387 | | 840 | 172600 |
| 338 | | 742 | 133841 | 388 | | 842 | 173441 |
| 339 | | 744 | 134584 | 389 | | 844 | 174284 |
| 340 | | 746 | 135329 | 390 | | 846 | 175129 |
| 341 | | 748 | 136076 | 391 | | 848 | 175976 |
| 342 | | 750 | 136825 | 392 | | 850 | 176825 |
| 343 | | 752 | 137576 | 393 | | 852 | 177676 |
| 344 | | 754 | 138329 | 394 | | 854 | 178529 |
| 345 | | 756 | 139084 | 395 | | 856 | 179384 |
| 346 | | 758 | 139841 | 396 | | 858 | 180241 |
| 347 | | 760 | 140600 | 397 | | 860 | 181100 |
| 348 | | 762 | 141361 | 398 | | 862 | 181961 |
| 349 | | 764 | 142124 | 399 | | 864 | 182824 |
| 350 | | 766 | 142889 | 400 | | 866 | 183689 |

| Cooper's *E*50. 0′–50′ | | | | | Cooper's *E*50. 50′–100′ | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Length | Wheel | Load | Load Sums | Moment Sums | Length | Wheel | Load | Load Sums | Moment Sums |
| 0 | w. 1 | 12.50 | 12.50 | 00.00 | 50 | ...... | .... | ...... | 4725.00 |
| 1 | .... | ..... | ...... | 12.50 | 51 | ...... | ..... | ...... | 4902.50 |
| 2 | .... | ..... | ...... | 25.00 | 52 | ...... | ..... | ...... | 5080.00 |
| 3 | .... | ..... | ...... | 37.50 | 53 | ...... | ..... | ...... | 5257.50 |
| 4 | .... | ..... | ...... | 50.00 | 54 | ...... | ..... | ...... | 5435.00 |
| 5 | .... | ..... | ...... | 62.50 | 55 | ...... | ..... | ...... | 5612.50 |
| 6 | .... | ..... | ...... | 75.00 | 56 | w. 10 | 12.50 | 190.00 | 5790.00 |
| 7 | .... | ..... | ...... | 87.50 | 57 | ...... | ..... | ...... | 5980.00 |
| 8 | w. 2 | 25.00 | 37.50 | 100.00 | 58 | ...... | ..... | ...... | 6170.00 |
| 9 | .... | ..... | ...... | 137.50 | 59 | ...... | ..... | ...... | 6360.00 |
| 10 | .... | ..... | ...... | 175.00 | 60 | ...... | ..... | ...... | 6550.00 |
| 11 | .... | ..... | ...... | 212.50 | 61 | ...... | ..... | ...... | 6740.00 |
| 12 | .... | ..... | ...... | 250.00 | 62 | ...... | ..... | ...... | 6930.00 |
| 13 | w. 3 | 25.00 | 62.50 | 287.50 | 63 | ...... | ..... | ...... | 7120.00 |
| 14 | .... | ..... | ...... | 350.00 | 64 | w. 11 | 25.00 | 215.00 | 7310.00 |
| 15 | .... | ..... | ...... | 412.50 | 65 | ...... | ..... | ...... | 7525.00 |
| 16 | .... | ..... | ...... | 475.00 | 66 | ...... | ..... | ...... | 7740.00 |
| 17 | .... | ..... | ...... | 537.50 | 67 | ...... | ..... | ...... | 7955.00 |
| 18 | w. 4 | 25.00 | 87.50 | 600.00 | 68 | ...... | ..... | ...... | 8170.00 |
| 19 | .... | ..... | ...... | 687.50 | 69 | w. 12 | 25.00 | 240.00 | 8385.00 |
| 20 | .... | ..... | ...... | 775.00 | 70 | ...... | ..... | ...... | 8625.00 |
| 21 | .... | ..... | ...... | 862.50 | 71 | ...... | ..... | ...... | 8865.00 |
| 22 | .... | ..... | ...... | 950.00 | 72 | ...... | ..... | ...... | 9105.00 |
| 23 | w. 5 | 25.00 | 112.50 | 1037.50 | 73 | ...... | ..... | ...... | 9345.00 |
| 24 | .... | ..... | ...... | 1150.00 | 74 | w. 13 | 25.00 | 265.00 | 9585.00 |
| 25 | .... | ..... | ...... | 1262.50 | 75 | ...... | ..... | ...... | 9850.00 |
| 26 | .... | ..... | ...... | 1375.00 | 76 | ...... | ..... | ...... | 10115.00 |
| 27 | .... | ..... | ...... | 1487.50 | 77 | ...... | ..... | ...... | 10380.00 |
| 28 | .... | ..... | ...... | 1600.00 | 78 | ...... | ..... | ...... | 10645.00 |
| 29 | .... | ..... | ...... | 1712.50 | 79 | w. 14 | 25.00 | 290.00 | 10910.00 |
| 30 | .... | ..... | ...... | 1825.00 | 80 | ...... | ..... | ...... | 11200.00 |
| 31 | .... | ..... | ...... | 1937.50 | 81 | ...... | ..... | ...... | 11490.00 |
| 32 | w. 6 | 16.25 | 128.75 | 2050.00 | 82 | ...... | ..... | ...... | 11780.00 |
| 33 | .... | ..... | ...... | 2178.75 | 83 | ...... | ..... | ...... | 12070.00 |
| 34 | .... | ..... | ...... | 2307.50 | 84 | ...... | ..... | ...... | 12360.00 |
| 35 | .... | ..... | ...... | 2436.25 | 85 | ...... | ..... | ...... | 12650.00 |
| 36 | .... | ..... | ...... | 2565.00 | 86 | ...... | ..... | ...... | 12940.00 |
| 37 | w. 7 | 16.25 | 145.00 | 2693.75 | 87 | ...... | ..... | ...... | 13230.00 |
| 38 | .... | ..... | ...... | 2838.75 | 88 | w. 15 | 16.25 | 306.25 | 13520.00 |
| 39 | .... | ..... | ...... | 2983.75 | 89 | ...... | ..... | ...... | 13826.25 |
| 40 | .... | ..... | ...... | 3128.75 | 90 | ...... | ..... | ...... | 14132.50 |
| 41 | .... | ..... | ...... | 3273.75 | 91 | ...... | ..... | ...... | 14438.75 |
| 42 | .... | ..... | ...... | 3418.75 | 92 | ...... | ..... | ...... | 14745.00 |
| 43 | w. 8 | 16.25 | 161.25 | 3563.75 | 93 | w. 16 | 16.25 | 322.50 | 15051.25 |
| 44 | .... | ..... | ...... | 3725.00 | 94 | ...... | ..... | ...... | 15373.75 |
| 45 | .... | ..... | ...... | 3886.25 | 95 | ...... | ..... | ...... | 15696.25 |
| 46 | .... | ..... | ...... | 4047.50 | 96 | ...... | ..... | ...... | 16018.75 |
| 47 | .... | ..... | ...... | 4208.75 | 97 | ...... | ..... | ...... | 16341.25 |
| 48 | w. 9 | 16.25 | 177.50 | 4370.00 | 98 | ...... | ..... | ...... | 16663.75 |
| 49 | .... | ..... | ...... | 4547.50 | 99 | w. 17 | 16.25 | 338.75 | 16986.25 |
| 50 | .... | ..... | ...... | 4725.00 | 100 | ...... | ..... | ...... | 17325.00 |

COOPER'S *E*50. 100′–150′

| Length | Wheel | Load | Load Sums | Moment Sums |
|---|---|---|---|---|
| 100 | ...... | ..... | ...... | 17325.00 |
| 101 | ...... | ..... | ...... | 17663.75 |
| 102 | ...... | ..... | ...... | 18002.50 |
| 103 | ...... | ..... | ...... | 18341.25 |
| 104 | w. 18 | 16.25 | 355.00 | 18680.00 |
| 105 | ...... | ..... | ...... | 19035.00 |
| 106 | ...... | ..... | ...... | 19390.00 |
| 107 | ...... | ..... | ...... | 19745.00 |
| 108 | ...... | ..... | ...... | 20100.00 |
| 109 | ...... | ..... | 355.00 | 20455.00 |
| 110 | ...... | Uniform Load = 2,500 pounds per foot | 357.50 | 20811.25 |
| 111 | ...... | | 360.00 | 21170.00 |
| 112 | ...... | | 362.50 | 21531.25 |
| 113 | ...... | | 365.00 | 21895.00 |
| 114 | ...... | | 367.50 | 22261.25 |
| 115 | ...... | | 370.00 | 22630.00 |
| 116 | ...... | | 372.50 | 23001.25 |
| 117 | ...... | | 375.00 | 23375.00 |
| 118 | ...... | | 377.50 | 23751.25 |
| 119 | ...... | | 380.00 | 24130.00 |
| 120 | ...... | | 382.50 | 24511.25 |
| 121 | ...... | | 385.00 | 24895.00 |
| 122 | ...... | | 387.50 | 25281.25 |
| 123 | ...... | | 390.00 | 25670.00 |
| 124 | ...... | | 392.50 | 26061.25 |
| 125 | ...... | | 395.00 | 26455.00 |
| 126 | ...... | | 397.50 | 26851.25 |
| 127 | ...... | | 400.00 | 27250.00 |
| 128 | ...... | | 402.50 | 27651.25 |
| 129 | ...... | | 405.00 | 28055.00 |
| 130 | ...... | | 407.50 | 28461.25 |
| 131 | ...... | | 410.00 | 28870.00 |
| 132 | ...... | | 412.50 | 29281.25 |
| 133 | ...... | | 415.00 | 29695.00 |
| 134 | ...... | | 417.50 | 30111.25 |
| 135 | ...... | | 420.00 | 30530.00 |
| 136 | ...... | | 422.50 | 30951.25 |
| 137 | ...... | | 425.00 | 31375.00 |
| 138 | ...... | | 427.50 | 31801.25 |
| 139 | ...... | | 430.00 | 32230.00 |
| 140 | ...... | | 432.50 | 32661.25 |
| 141 | ...... | | 435.00 | 33095.00 |
| 142 | ...... | | 437.50 | 33531.25 |
| 143 | ...... | | 440.00 | 33970.00 |
| 144 | ...... | | 442.50 | 34411.00 |
| 145 | ...... | | 445.00 | 34855.00 |
| 146 | ...... | | 447.50 | 35301.25 |
| 147 | ...... | | 450.00 | 35750.00 |
| 148 | ...... | | 452.50 | 36201.25 |
| 149 | ...... | | 455.00 | 36655.00 |
| 150 | ...... | | 457.50 | 37111.25 |

COOPER'S *E*50. 150′-200′

| Length | Load | Load Sums | Moment Sums |
|---|---|---|---|
| 150 | Uniform Load = 2,500 pounds per foot | 457.50 | 37111.25 |
| 151 | | 460.00 | 37570.00 |
| 152 | | 462.50 | 38031.25 |
| 153 | | 465.00 | 38495.00 |
| 154 | | 467.50 | 38961.25 |
| 155 | | 470.00 | 39430.00 |
| 156 | | 472.50 | 39901.25 |
| 157 | | 475.00 | 40375.00 |
| 158 | | 477.50 | 40851.25 |
| 159 | | 480.00 | 41330.00 |
| 160 | | 482.50 | 41811.25 |
| 161 | | 485.00 | 42295.00 |
| 162 | | 487.50 | 42781.25 |
| 163 | | 490.00 | 43270.00 |
| 164 | | 492.50 | 43761.25 |
| 165 | | 495.00 | 44255.00 |
| 166 | | 497.50 | 44751.25 |
| 167 | | 500.00 | 45250.00 |
| 168 | | 502.50 | 45751.25 |
| 169 | | 505.00 | 46255.00 |
| 170 | | 507.50 | 46761.25 |
| 171 | | 510.00 | 47270.00 |
| 172 | | 512.50 | 47781.25 |
| 173 | | 515.00 | 48295.00 |
| 174 | | 517.50 | 48811.25 |
| 175 | | 520.00 | 49330.00 |
| 176 | | 522.50 | 49851.25 |
| 177 | | 525.00 | 50375.00 |
| 178 | | 527.50 | 50901.25 |
| 179 | | 530.00 | 51430.00 |
| 180 | | 532.50 | 51961.25 |
| 181 | | 535.00 | 52495.00 |
| 182 | | 537.50 | 53031.25 |
| 183 | | 540.00 | 53570.00 |
| 184 | | 542.50 | 54111.25 |
| 185 | | 545.00 | 54655.00 |
| 186 | | 547.50 | 55201.25 |
| 187 | | 550.00 | 55750.00 |
| 188 | | 552.50 | 56301.25 |
| 189 | | 555.00 | 56855.00 |
| 190 | | 557.50 | 57411.25 |
| 191 | | 560.00 | 57970.00 |
| 192 | | 562.50 | 58531.25 |
| 193 | | 565.00 | 59095.00 |
| 194 | | 567.50 | 59661.25 |
| 195 | | 570.00 | 60230.00 |
| 196 | | 572.50 | 60801.25 |
| 197 | | 575.00 | 61375.00 |
| 198 | | 577.50 | 61951.25 |
| 199 | | 580.00 | 62530.00 |
| 200 | | 582.50 | 63111.25 |

Cooper's $E50$. 200′–250′

| Length | Load | Load Sums | Moment Sums |
|---|---|---|---|
| 200 | Uniform Load = 2,500 pounds per foot | 582.50 | 63111.25 |
| 201 | | 585.00 | 63695.00 |
| 202 | | 587.50 | 64281.25 |
| 203 | | 590.00 | 64870.00 |
| 204 | | 592.50 | 65461.25 |
| 205 | | 595.00 | 66055.00 |
| 206 | | 597.50 | 66651.25 |
| 207 | | 600.00 | 67250.00 |
| 208 | | 602.50 | 67851.25 |
| 209 | | 605.00 | 68455.00 |
| 210 | | 607.50 | 69061.25 |
| 211 | | 610.00 | 69670.00 |
| 212 | | 612.50 | 70281.25 |
| 213 | | 615.00 | 70895.00 |
| 214 | | 617.50 | 71511.25 |
| 215 | | 620.00 | 72130.00 |
| 216 | | 622.50 | 72751.25 |
| 217 | | 625.00 | 73375.00 |
| 218 | | 627.50 | 74001.25 |
| 219 | | 630.00 | 74630.00 |
| 220 | | 632.50 | 75261.25 |
| 221 | | 635.00 | 75895.00 |
| 222 | | 637.50 | 76531.25 |
| 223 | | 640.00 | 77170.00 |
| 224 | | 642.50 | 77811.25 |
| 225 | | 645.00 | 78455.00 |
| 226 | | 647.50 | 79101.25 |
| 227 | | 650.00 | 79750.00 |
| 228 | | 652.50 | 80401.25 |
| 229 | | 655.00 | 81055.00 |
| 230 | | 657.50 | 81711.25 |
| 231 | | 660.00 | 82370.00 |
| 232 | | 662.50 | 83031.25 |
| 233 | | 665.00 | 83695.00 |
| 234 | | 667.50 | 84361.25 |
| 235 | | 670.00 | 85030.00 |
| 236 | | 672.50 | 85701.25 |
| 237 | | 675.00 | 86375.00 |
| 238 | | 677.50 | 87051.25 |
| 239 | | 680.00 | 87730.00 |
| 240 | | 682.50 | 88411.25 |
| 241 | | 685.00 | 89095.00 |
| 242 | | 687.50 | 89781.25 |
| 243 | | 690.00 | 90470.00 |
| 244 | | 692.50 | 91161.25 |
| 245 | | 695.00 | 91855.00 |
| 246 | | 697.50 | 92551.25 |
| 247 | | 700.00 | 93250.00 |
| 248 | | 702.50 | 93951.25 |
| 249 | | 705.00 | 94655.00 |
| 250 | | 707.50 | 95361.25 |

Cooper's $E50$. 250′–300

| Length | Load | Load Sums | Moment Sums |
|---|---|---|---|
| 250 | Uniform Load = 2,500 pounds per foot | 707.50 | 95361.25 |
| 251 | | 710.00 | 96070.00 |
| 252 | | 712.50 | 96781.25 |
| 253 | | 715.00 | 97495.00 |
| 254 | | 717.50 | 98211.25 |
| 255 | | 720.00 | 98930.00 |
| 256 | | 722.50 | 99651.25 |
| 257 | | 725.00 | 100375.00 |
| 258 | | 727.50 | 101101.25 |
| 259 | | 730.00 | 101830.00 |
| 260 | | 732.50 | 102561.25 |
| 261 | | 735.00 | 103295.00 |
| 262 | | 737.50 | 104031.25 |
| 263 | | 740.00 | 104770.00 |
| 264 | | 742.50 | 105511.25 |
| 265 | | 745.00 | 106255.00 |
| 266 | | 747.50 | 107001.25 |
| 267 | | 750.00 | 107750.00 |
| 268 | | 752.50 | 108501.25 |
| 269 | | 755.00 | 109255.00 |
| 270 | | 757.50 | 110011.25 |
| 271 | | 760.00 | 110770.00 |
| 272 | | 762.50 | 111531.25 |
| 273 | | 765.00 | 112295.00 |
| 274 | | 767.50 | 113061.25 |
| 275 | | 770.00 | 113830.00 |
| 276 | | 772.50 | 114601.25 |
| 277 | | 775.00 | 115375.00 |
| 278 | | 777.50 | 116151.25 |
| 279 | | 780.00 | 116930.00 |
| 280 | | 782.50 | 117711.25 |
| 281 | | 785.00 | 118495.00 |
| 282 | | 787.50 | 119281.25 |
| 283 | | 790.00 | 120070.00 |
| 284 | | 792.50 | 120861.25 |
| 285 | | 795.00 | 121655.00 |
| 286 | | 797.50 | 122451.25 |
| 287 | | 800.00 | 123250.00 |
| 288 | | 802.50 | 124051.25 |
| 289 | | 805.00 | 124855.00 |
| 290 | | 807.50 | 125661.25 |
| 291 | | 810.00 | 126470.00 |
| 292 | | 812.50 | 127281.25 |
| 293 | | 815.00 | 128095.00 |
| 294 | | 817.50 | 128911.25 |
| 295 | | 820.00 | 129730.00 |
| 296 | | 822.50 | 130551.25 |
| 297 | | 825.00 | 131375.00 |
| 298 | | 827.50 | 132201.25 |
| 299 | | 830.00 | 133030.00 |
| 300 | | 832.50 | 133861.25 |

COOPER'S *E*50. 300′–350′

| Length | Load | Load Sums | Moment Sums |
|---|---|---|---|
| 300 | Uniform Load = 2,500 pounds per foot | 832.50 | 133861.25 |
| 301 | | 835.00 | 134695.00 |
| 302 | | 837.50 | 135531.25 |
| 303 | | 840.00 | 136370.00 |
| 304 | | 842.50 | 137211.25 |
| 305 | | 845.00 | 138055.00 |
| 306 | | 847.50 | 138901.25 |
| 307 | | 850.00 | 139750.00 |
| 308 | | 852.50 | 140601.25 |
| 309 | | 855.00 | 141455.00 |
| 310 | | 857.50 | 142311.25 |
| 311 | | 860.00 | 143170.00 |
| 312 | | 862.50 | 144031.25 |
| 313 | | 865.00 | 144895.00 |
| 314 | | 867.50 | 145761.25 |
| 315 | | 870.00 | 146630.00 |
| 316 | | 872.50 | 147501.25 |
| 317 | | 875.00 | 148375.00 |
| 318 | | 877.50 | 149251.25 |
| 319 | | 880.00 | 150130.00 |
| 320 | | 882.50 | 151011.25 |
| 321 | | 885.00 | 151895.00 |
| 322 | | 887.50 | 152781.25 |
| 323 | | 890.00 | 153670.00 |
| 324 | | 892.50 | 154561.25 |
| 325 | | 895.00 | 155455.00 |
| 326 | | 897.50 | 156351.25 |
| 327 | | 900.00 | 157250.00 |
| 328 | | 902.50 | 158151.25 |
| 329 | | 905.00 | 159055.00 |
| 330 | | 907.50 | 159961.25 |
| 331 | | 910.00 | 160870.00 |
| 332 | | 912.50 | 161781.25 |
| 333 | | 915.00 | 162695.00 |
| 334 | | 917.50 | 163611.25 |
| 335 | | 920.00 | 164530.00 |
| 336 | | 922.50 | 165451.25 |
| 337 | | 925.00 | 166375.00 |
| 338 | | 927.50 | 167301.25 |
| 339 | | 930.00 | 168230.00 |
| 340 | | 932.50 | 169161.25 |
| 341 | | 935.00 | 170095.00 |
| 342 | | 937.50 | 171031.25 |
| 343 | | 940.00 | 171970.00 |
| 344 | | 942.50 | 172911.25 |
| 345 | | 945.00 | 173855.00 |
| 346 | | 947.50 | 174801.25 |
| 347 | | 950.00 | 175750.00 |
| 348 | | 952.50 | 176701.25 |
| 349 | | 955.00 | 177655.00 |
| 350 | | 957.50 | 178611.25 |

COOPER'S *E*50. 350′–400′

| Length | Load | Load Sums | Moment Sums |
|---|---|---|---|
| 350 | Uniform Load = 2,500 pounds per foot | 957.50 | 178611.25 |
| 351 | | 960.00 | 179570.00 |
| 352 | | 962.50 | 180531.25 |
| 353 | | 965.00 | 181495.00 |
| 354 | | 967.50 | 182461.25 |
| 355 | | 970.00 | 183430.00 |
| 356 | | 972.50 | 184401.25 |
| 357 | | 975.00 | 185375.00 |
| 358 | | 977.50 | 186351.25 |
| 359 | | 980.00 | 187330.00 |
| 360 | | 982.50 | 188311.25 |
| 361 | | 985.00 | 189295.00 |
| 362 | | 987.50 | 190281.25 |
| 363 | | 990.00 | 191270.00 |
| 364 | | 992.50 | 192261.25 |
| 365 | | 995.00 | 193255.00 |
| 366 | | 997.50 | 194251.25 |
| 367 | | 1000.00 | 195250.00 |
| 368 | | 1002.50 | 196251.25 |
| 369 | | 1005.00 | 197255.00 |
| 370 | | 1007.50 | 198261.25 |
| 371 | | 1010.00 | 199270.00 |
| 372 | | 1012.50 | 200281.25 |
| 373 | | 1015.00 | 201295.00 |
| 374 | | 1017.50 | 202311.25 |
| 375 | | 1020.00 | 203330.00 |
| 376 | | 1022.50 | 204351.25 |
| 377 | | 1025.00 | 205375.00 |
| 378 | | 1027.50 | 206401.25 |
| 379 | | 1030.00 | 207430.00 |
| 380 | | 1032.50 | 208461.25 |
| 381 | | 1035.00 | 209495.00 |
| 382 | | 1037.50 | 210531.25 |
| 383 | | 1040.00 | 211570.00 |
| 384 | | 1042.50 | 212611.25 |
| 385 | | 1045.00 | 213655.00 |
| 386 | | 1047.50 | 214701.25 |
| 387 | | 1050.00 | 215750.00 |
| 388 | | 1052.50 | 216801.25 |
| 389 | | 1055.00 | 217855.00 |
| 390 | | 1057.50 | 218911.25 |
| 391 | | 1060.00 | 219970.00 |
| 392 | | 1062.50 | 221031.25 |
| 393 | | 1065.00 | 222095.00 |
| 394 | | 1067.50 | 223161.25 |
| 395 | | 1070.00 | 224230.00 |
| 396 | | 1072.50 | 225301.25 |
| 397 | | 1075.00 | 226375.00 |
| 398 | | 1077.50 | 227451.25 |
| 399 | | 1080.00 | 228530.00 |
| 400 | | 1082.50 | 229611.25 |

Cooper's *E*60. 0′–50′ | Cooper's *E*60. 50′–100′

| Length | Wheel | Load | Load Sums | Moment Sums | Length | Wheel | Load | Load Sums | Moment Sums |
|---|---|---|---|---|---|---|---|---|---|
| 0 | w. 1 | 15.0 | 15.0 | 00.00 | 50 | ...... | .... | ..... | 5670.00 |
| 1 | .... | .... | ..... | 15.00 | 51 | ...... | .... | ..... | 5883.00 |
| 2 | .... | .... | ..... | 30.00 | 52 | ...... | .... | ..... | 6096.00 |
| 3 | .... | .... | ..... | 45.00 | 53 | ...... | .... | ..... | 6309.00 |
| 4 | .... | .... | ..... | 60.00 | 54 | ...... | .... | ..... | 6522.00 |
| 5 | .... | .... | ..... | 75.00 | 55 | ...... | .... | ..... | 6735.00 |
| 6 | .... | .... | ..... | 90.00 | 56 | w. 10 | 15.0 | 228.0 | 6948.00 |
| 7 | .... | .... | ..... | 105.00 | 57 | ...... | .... | ..... | 7176.00 |
| 8 | w. 2 | 30.0 | 45.0 | 120.00 | 58 | ...... | .... | ..... | 7404.00 |
| 9 | .... | .... | ..... | 165.00 | 59 | ...... | .... | ..... | 7632.00 |
| 10 | .... | .... | ..... | 210.00 | 60 | ...... | .... | ..... | 7860.00 |
| 11 | .... | .... | ..... | 255.00 | 61 | ...... | .... | ..... | 8088.00 |
| 12 | .... | .... | ..... | 300.00 | 62 | ...... | .... | ..... | 8316.00 |
| 13 | w. 3 | 30.0 | 75.0 | 345.00 | 63 | ...... | .... | ..... | 8544.00 |
| 14 | .... | .... | ..... | 420.00 | 64 | w. 11 | 30.0 | 258.0 | 8772.00 |
| 15 | .... | .... | ..... | 495.00 | 65 | ...... | .... | ..... | 9030.00 |
| 16 | .... | .... | ..... | 570.00 | 66 | ...... | .... | ..... | 9288.00 |
| 17 | .... | .... | ..... | 645.00 | 67 | ...... | .... | ..... | 9546.00 |
| 18 | w. 4 | 30.0 | 105.0 | 720.00 | 68 | ...... | .... | ..... | 9804.00 |
| 19 | .... | .... | ..... | 825.00 | 69 | w. 12 | 30.0 | 288.0 | 10062.00 |
| 20 | .... | .... | ..... | 930.00 | 70 | ...... | .... | ..... | 10350.00 |
| 21 | .... | .... | ..... | 1035.00 | 71 | ...... | .... | ..... | 10638.00 |
| 22 | .... | .... | ..... | 1140.00 | 72 | ...... | .... | ..... | 10926.00 |
| 23 | w. 5 | 30.0 | 135.0 | 1245.00 | 73 | ...... | .... | ..... | 11214.00 |
| 24 | .... | .... | ..... | 1380.00 | 74 | w. 13 | 30.0 | 318.0 | 11502.00 |
| 25 | .... | .... | ..... | 1515.00 | 75 | ...... | .... | ..... | 11820.00 |
| 26 | .... | .... | ..... | 1650.00 | 76 | ...... | .... | ..... | 12138.00 |
| 27 | .... | .... | ..... | 1785.00 | 77 | ...... | .... | ..... | 12456.00 |
| 28 | .... | .... | ..... | 1920.00 | 78 | ...... | .... | ..... | 12774.00 |
| 29 | .... | .... | ..... | 2055.00 | 79 | w. 14 | 30.0 | 348.0 | 13092.00 |
| 30 | .... | .... | ..... | 2190.00 | 80 | ...... | .... | ..... | 13440.00 |
| 31 | .... | .... | ..... | 2325.00 | 81 | ...... | .... | ..... | 13788.00 |
| 32 | w. 6 | 19.5 | 154.5 | 2460.00 | 82 | ...... | .... | ..... | 14136.00 |
| 33 | .... | .... | ..... | 2614.50 | 83 | ...... | .... | ..... | 14484.00 |
| 34 | .... | .... | ..... | 2769.00 | 84 | ...... | .... | ..... | 14832.00 |
| 35 | .... | .... | .... | 2923.50 | 85 | ...... | .... | ..... | 15180.00 |
| 36 | .... | .... | ..... | 3078.00 | 86 | ...... | .... | ..... | 15528.00 |
| 37 | w. 7 | 19.5 | 174.0 | 3232.50 | 87 | ...... | .... | ..... | 15876.00 |
| 38 | .... | .... | ..... | 3406.50 | 88 | w. 15 | 19.5 | 367.5 | 16224.00 |
| 39 | .... | .... | ..... | 3580.50 | 89 | ...... | .... | ..... | 16591.00 |
| 40 | .... | .... | ..... | 3754.50 | 90 | ...... | .... | ..... | 16959.00 |
| 41 | .... | .... | ..... | 3928.50 | 91 | ...... | .... | ..... | 17326.50 |
| 42 | .... | .... | ..... | 4102.50 | 92 | ...... | .... | ..... | 17694.00 |
| 43 | w. 8 | 19.5 | 193.5 | 4276.50 | 93 | w. 16 | 19.5 | 387.0 | 18061.50 |
| 44 | .... | .... | ..... | 4470.00 | 94 | ...... | .... | .... | 18448.00 |
| 45 | .... | .... | ..... | 4663.50 | 95 | ...... | .... | ..... | 18835.50 |
| 46 | .... | .... | ..... | 4857.00 | 96 | ...... | .... | ..... | 19222.50 |
| 47 | .... | .... | ..... | 5050.50 | 97 | ...... | .... | ..... | 19609.50 |
| 48 | w. 9 | 19.5 | 213.0 | 5244.00 | 98 | ...... | .... | ..... | 19996.50 |
| 49 | .... | .... | ..... | 5457.00 | 99 | w. 17 | 19.5 | 406.5 | 20383.50 |
| 50 | .... | .... | ..... | 5670.00 | 100 | ...... | .... | ..... | 20790.00 |

Cooper's $E$60. 100′–150′

| Length | Wheel | Load | Load Sums | Moment Sums |
|---|---|---|---|---|
| 100 | ...... | .... | ..... | 20790.00 |
| 101 | ...... | .... | ..... | 21196.50 |
| 102 | ...... | .... | ..... | 21603.00 |
| 103 | ...... | .... | ..... | 22009.50 |
| 104 | w. 18 | 19.5 | 426.0 | 22416.00 |
| 105 | ...... | .... | ..... | 22842.00 |
| 106 | ...... | .... | ..... | 23268.00 |
| 107 | ...... | .... | ..... | 23694.00 |
| 108 | ...... | .... | ..... | 24120.00 |
| 109 | | | 426.0 | 24546.00 |
| 110 | Uniform Load = 3,000 pounds per foot | | 429.0 | 24973.50 |
| 111 | | | 432.0 | 25404.00 |
| 112 | | | 435.0 | 25837.50 |
| 113 | | | 438.0 | 26274.00 |
| 114 | | | 441.0 | 26713.50 |
| 115 | | | 444.0 | 27156.00 |
| 116 | | | 447.0 | 27601.50 |
| 117 | | | 450.0 | 28050.00 |
| 118 | | | 453.0 | 28501.50 |
| 119 | | | 456.0 | 28956.00 |
| 120 | | | 459.0 | 29413.50 |
| 121 | | | 462.0 | 29874.00 |
| 122 | | | 465.0 | 30337.50 |
| 123 | | | 468.0 | 30804.00 |
| 124 | | | 471.0 | 31273.50 |
| 125 | | | 474.0 | 31746.00 |
| 126 | | | 477.0 | 32221.50 |
| 127 | | | 480.0 | 32700.00 |
| 128 | | | 483.0 | 33181.50 |
| 129 | | | 486.0 | 33666.00 |
| 130 | | | 489.0 | 34153.50 |
| 131 | | | 492.0 | 34644.00 |
| 132 | | | 495.0 | 35137.50 |
| 133 | | | 498.0 | 35634.00 |
| 134 | | | 501.0 | 36133.50 |
| 135 | | | 504.0 | 36636.00 |
| 136 | | | 507.0 | 37141.50 |
| 137 | | | 510.0 | 37650.00 |
| 138 | | | 513.0 | 38161.50 |
| 139 | | | 516.0 | 38676.00 |
| 140 | | | 519.0 | 39193.50 |
| 141 | | | 522.0 | 39714.00 |
| 142 | | | 525.0 | 40237.50 |
| 143 | | | 528.0 | 40764.00 |
| 144 | | | 531.0 | 41293.50 |
| 145 | | | 534.0 | 41826.00 |
| 146 | | | 537.0 | 42361.50 |
| 147 | | | 540.0 | 42900.00 |
| 148 | | | 543.0 | 43441.50 |
| 149 | | | 546.0 | 43986.00 |
| 150 | | | 549.0 | 44533.50 |

Cooper's $E$60. 150′–200′

| Length | Load | Load Sums | Moment Sums |
|---|---|---|---|
| 150 | Uniform Load = 3,000 pounds per foot | 549.0 | 44533.50 |
| 151 | | 552.0 | 45084.00 |
| 152 | | 555.0 | 45637.50 |
| 153 | | 558.0 | 46194.00 |
| 154 | | 561.0 | 46753.50 |
| 155 | | 564.0 | 47316.00 |
| 156 | | 567.0 | 47881.50 |
| 157 | | 570.0 | 48450.00 |
| 158 | | 573.0 | 49021.50 |
| 159 | | 576.0 | 49596.00 |
| 160 | | 579.0 | 50173.50 |
| 161 | | 582.0 | 50754.00 |
| 162 | | 585.0 | 51337.50 |
| 163 | | 588.0 | 51924.00 |
| 164 | | 591.0 | 52513.50 |
| 165 | | 594.0 | 53106.00 |
| 166 | | 597.0 | 53701.50 |
| 167 | | 600.0 | 54300.00 |
| 168 | | 603.0 | 54901.50 |
| 169 | | 606.0 | 55506.00 |
| 170 | | 609.0 | 56113.50 |
| 171 | | 612.0 | 56724.00 |
| 172 | | 615.0 | 57337.50 |
| 173 | | 618.0 | 57954.00 |
| 174 | | 621.0 | 58573.50 |
| 175 | | 624.0 | 59196.00 |
| 176 | | 627.0 | 59821.50 |
| 177 | | 630.0 | 60450.00 |
| 178 | | 633.0 | 61081.50 |
| 179 | | 636.0 | 61716.00 |
| 180 | | 639.0 | 62353.50 |
| 181 | | 642.0 | 62994.00 |
| 182 | | 645.0 | 63637.50 |
| 183 | | 648.0 | 64284.00 |
| 184 | | 651.0 | 64933.50 |
| 185 | | 654.0 | 65586.00 |
| 186 | | 657.0 | 66241.50 |
| 187 | | 660.0 | 66900.00 |
| 188 | | 663.0 | 67561.50 |
| 189 | | 666.0 | 68226.00 |
| 190 | | 669.0 | 68893.50 |
| 191 | | 672.0 | 69564.00 |
| 192 | | 675.0 | 70237.50 |
| 193 | | 678.0 | 70914.00 |
| 194 | | 681.0 | 71593.50 |
| 195 | | 684.0 | 72276.00 |
| 196 | | 687.0 | 72961.50 |
| 197 | | 690.0 | 73650.00 |
| 198 | | 693.0 | 74341.50 |
| 199 | | 696.0 | 75036.00 |
| 200 | | 699.0 | 75733.50 |

| Cooper's *E*60. 200′–250′ | | | | Cooper's *E*60. 250′–300′ | | | |
|---|---|---|---|---|---|---|---|
| Length | Load | Load Sums | Moment Sums | Length | Load | Load Sums | Moment Sums |
| 200 | Uniform Load = 3,000 pounds per foot | 699.0 | 75733.50 | 250 | Uniform Load = 3,000 pounds per foot | 849.0 | 114433.50 |
| 201 | | 702.0 | 76434.00 | 251 | | 852.0 | 115284.00 |
| 202 | | 705.0 | 77137.50 | 252 | | 855.0 | 116137.50 |
| 203 | | 708.0 | 77844.00 | 253 | | 858.0 | 116994.00 |
| 204 | | 711.0 | 78553.50 | 254 | | 861.0 | 117853.50 |
| 205 | | 714.0 | 79266.00 | 255 | | 864.0 | 118716.00 |
| 206 | | 717.0 | 79981.50 | 256 | | 867.0 | 119581.50 |
| 207 | | 720.0 | 80700.00 | 257 | | 870.0 | 120450.00 |
| 208 | | 723.0 | 81421.50 | 258 | | 873.0 | 121321.50 |
| 209 | | 726.0 | 82146.00 | 259 | | 876.0 | 122196.00 |
| 210 | | 729.0 | 82873.50 | 260 | | 879.0 | 123073.50 |
| 211 | | 732.0 | 83604.00 | 261 | | 882.0 | 123954.00 |
| 212 | | 735.0 | 84337.50 | 262 | | 885.0 | 124837.50 |
| 213 | | 738.0 | 85074.00 | 263 | | 888.0 | 125724.00 |
| 214 | | 741.0 | 85813.50 | 264 | | 891.0 | 126613.50 |
| 215 | | 744.0 | 86556.00 | 265 | | 894.0 | 127506.00 |
| 216 | | 747.0 | 87301.50 | 266 | | 897.0 | 128401.50 |
| 217 | | 750.0 | 88050.00 | 267 | | 900.0 | 129300.00 |
| 218 | | 753.0 | 88801.50 | 268 | | 903.0 | 130201.50 |
| 219 | | 756.0 | 89556.00 | 269 | | 906.0 | 131106.00 |
| 220 | | 759.0 | 90313.50 | 270 | | 909.0 | 132013.50 |
| 221 | | 762.0 | 91074.00 | 271 | | 912.0 | 132924.00 |
| 222 | | 765.0 | 91837.50 | 272 | | 915.0 | 133837.50 |
| 223 | | 768.0 | 92604.00 | 273 | | 918.0 | 134754.00 |
| 224 | | 771.0 | 93373.50 | 274 | | 921.0 | 135673.50 |
| 225 | | 774.0 | 94146.00 | 275 | | 924.0 | 136596.00 |
| 226 | | 777.0 | 94921.50 | 276 | | 927.0 | 137521.50 |
| 227 | | 780.0 | 95700.00 | 277 | | 930.0 | 138450.00 |
| 228 | | 783.0 | 96481.50 | 278 | | 933.0 | 139381.50 |
| 229 | | 786.0 | 97266.00 | 279 | | 936.0 | 140316.00 |
| 230 | | 789.0 | 98053.50 | 280 | | 939.0 | 141253.50 |
| 231 | | 792.0 | 98844.00 | 281 | | 942.0 | 142194.00 |
| 232 | | 795.0 | 99637.50 | 282 | | 945.0 | 143137.50 |
| 233 | | 798.0 | 100434.00 | 283 | | 948.0 | 144084.00 |
| 234 | | 801.0 | 101233.50 | 284 | | 951.0 | 145033.50 |
| 235 | | 804.0 | 102036.00 | 285 | | 954.0 | 145986.00 |
| 236 | | 807.0 | 102841.50 | 286 | | 957.0 | 146941.50 |
| 237 | | 810.0 | 103650.00 | 287 | | 960.0 | 147900.00 |
| 238 | | 813.0 | 104461.50 | 288 | | 963.0 | 148861.50 |
| 239 | | 816.0 | 105276.00 | 289 | | 966.0 | 149826.00 |
| 240 | | 819.0 | 106093.50 | 290 | | 969.0 | 150793.50 |
| 241 | | 822.0 | 106914.00 | 291 | | 972.0 | 151764.00 |
| 242 | | 825.0 | 107737.50 | 292 | | 975.0 | 152737.50 |
| 243 | | 828.0 | 108564.00 | 293 | | 978.0 | 153714.00 |
| 244 | | 831.0 | 109393.50 | 294 | | 981.0 | 154693.50 |
| 245 | | 834.0 | 110226.00 | 295 | | 984.0 | 155676.00 |
| 246 | | 837.0 | 111061.50 | 296 | | 987.0 | 156661.50 |
| 247 | | 840.0 | 111900.00 | 297 | | 990.0 | 157650.00 |
| 248 | | 843.0 | 112741.50 | 298 | | 993.0 | 158641.50 |
| 249 | | 846.0 | 113586.00 | 299 | | 996.0 | 159636.00 |
| 250 | | 849.0 | 114433.50 | 300 | | 999.0 | 160633.50 |

Cooper's $E$60. 300′–350′

| Length | Load | Load Sums | Moment Sums |
|---|---|---|---|
| 300 | Uniform Load = 3,000 pounds per foot | 999.0 | 160633.50 |
| 301 | | 1002.0 | 161634.00 |
| 302 | | 1005.0 | 162637.50 |
| 303 | | 1008.0 | 163644.00 |
| 304 | | 1011.0 | 164653.50 |
| 305 | | 1014.0 | 165666.00 |
| 306 | | 1017.0 | 166681.50 |
| 307 | | 1020.0 | 167700.00 |
| 308 | | 1023.0 | 168721.50 |
| 309 | | 1026.0 | 169746.00 |
| 310 | | 1029.0 | 170773.50 |
| 311 | | 1032.0 | 171804.00 |
| 312 | | 1035.0 | 172837.50 |
| 313 | | 1038.0 | 173874.00 |
| 314 | | 1041.0 | 174913.50 |
| 315 | | 1044.0 | 175956.00 |
| 316 | | 1047.0 | 177001.50 |
| 317 | | 1050.0 | 178050.00 |
| 318 | | 1053.0 | 179101.50 |
| 319 | | 1056.0 | 180156.00 |
| 320 | | 1059.0 | 181213.50 |
| 321 | | 1062.0 | 182274.00 |
| 322 | | 1065.0 | 183337.50 |
| 323 | | 1068.0 | 184404.00 |
| 324 | | 1071.0 | 185473.50 |
| 325 | | 1074.0 | 186546.00 |
| 326 | | 1077.0 | 187621.50 |
| 327 | | 1080.0 | 188700.00 |
| 328 | | 1083.0 | 189781.50 |
| 329 | | 1086.0 | 190866.00 |
| 330 | | 1089.0 | 191953.50 |
| 331 | | 1092.0 | 193044.00 |
| 332 | | 1095.0 | 194137.50 |
| 333 | | 1098.0 | 195234.00 |
| 334 | | 1101.0 | 196333.50 |
| 335 | | 1104.0 | 197436.00 |
| 336 | | 1107.0 | 198541.50 |
| 337 | | 1110.0 | 199650.00 |
| 338 | | 1113.0 | 200761.50 |
| 339 | | 1116.0 | 201876.00 |
| 340 | | 1119.0 | 202993.50 |
| 341 | | 1122.0 | 204114.00 |
| 342 | | 1125.0 | 205237.50 |
| 343 | | 1128.0 | 206364.00 |
| 344 | | 1131.0 | 207493.50 |
| 345 | | 1134.0 | 208626.00 |
| 346 | | 1137.0 | 209761.50 |
| 347 | | 1140.0 | 210900.00 |
| 348 | | 1143.0 | 212041.50 |
| 349 | | 1146.0 | 213186.00 |
| 350 | | 1149.0 | 214333.50 |

Cooper's $E$60. 350′–400′

| Length | Load | Load Sums | Moment Sums |
|---|---|---|---|
| 350 | Uniform Load = 3,000 pounds per foot | 1149.0 | 214333.50 |
| 351 | | 1152.0 | 215484.00 |
| 352 | | 1155.0 | 216637.50 |
| 353 | | 1158.0 | 217794.00 |
| 354 | | 1161.0 | 218953.50 |
| 355 | | 1164.0 | 220116.00 |
| 356 | | 1167.0 | 221281.50 |
| 357 | | 1170.0 | 222450.00 |
| 358 | | 1173.0 | 223621.50 |
| 359 | | 1176.0 | 224796.00 |
| 360 | | 1179.0 | 225973.50 |
| 361 | | 1182.0 | 227154.00 |
| 362 | | 1185.0 | 228337.50 |
| 363 | | 1188.0 | 229524.00 |
| 364 | | 1191.0 | 230713.50 |
| 365 | | 1194.0 | 231906.00 |
| 366 | | 1197.0 | 233101.50 |
| 367 | | 1200.0 | 234300.00 |
| 368 | | 1203.0 | 235501.50 |
| 369 | | 1206.0 | 236706.00 |
| 370 | | 1209.0 | 237913.50 |
| 371 | | 1212.0 | 239124.00 |
| 372 | | 1215.0 | 240337.50 |
| 373 | | 1218.0 | 241554.00 |
| 374 | | 1221.0 | 242773.50 |
| 375 | | 1224.0 | 243996.00 |
| 376 | | 1227.0 | 245221.50 |
| 377 | | 1230.0 | 246450.00 |
| 378 | | 1233.0 | 247681.50 |
| 379 | | 1236.0 | 248916.00 |
| 380 | | 1239.0 | 250153.50 |
| 381 | | 1242.0 | 251394.00 |
| 382 | | 1245.0 | 252637.50 |
| 383 | | 1248.0 | 253884.00 |
| 384 | | 1251.0 | 255133.50 |
| 385 | | 1254.0 | 256386.00 |
| 386 | | 1257.0 | 257641.50 |
| 387 | | 1260.0 | 258900.00 |
| 388 | | 1263.0 | 260161.50 |
| 389 | | 1266.0 | 261426.00 |
| 390 | | 1269.0 | 262693.50 |
| 391 | | 1272.0 | 263964.00 |
| 392 | | 1275.0 | 265237.50 |
| 393 | | 1278.0 | 266514.00 |
| 394 | | 1281.0 | 267793.50 |
| 395 | | 1284.0 | 269076.00 |
| 396 | | 1287.0 | 270361.50 |
| 397 | | 1290.0 | 271650.00 |
| 398 | | 1293.0 | 272941.50 |
| 399 | | 1296.0 | 274236.00 |
| 400 | | 1299.0 | 275533.50 |

COMMON STANDARD 0′–50′

| Length | Wheel | Load | Load Sums | Moment Sums |
|---|---|---|---|---|
| 0 | w. 1 | 12.5 | 12.5 | 00.00 |
| 1 | ...... | .... | ..... | 12.50 |
| 2 | ...... | .... | ..... | 25.00 |
| 3 | ...... | .... | ..... | 37.50 |
| 4 | ...... | .... | ..... | 50.00 |
| 5 | ...... | .... | ..... | 62.50 |
| 6 | ...... | .... | ..... | 75.00 |
| 7 | ...... | .... | ..... | 87.50 |
| 8 | w. 2 | 27.5 | 40.0 | 100.00 |
| 9 | ...... | .... | ..... | 140.00 |
| 10 | ...... | .... | ..... | 180.00 |
| 11 | ...... | .... | ..... | 220.00 |
| 12 | ...... | .... | ..... | 260.00 |
| 13 | w. 3 | 27.5 | 67.5 | 300.00 |
| 14 | ...... | .... | ..... | 367.50 |
| 15 | ...... | .... | ..... | 435.00 |
| 16 | ...... | .... | ..... | 502.50 |
| 17 | ...... | .... | ..... | 570.00 |
| 18 | w. 4 | 27.5 | 95.0 | 637.50 |
| 19 | ...... | .... | ..... | 732.50 |
| 20 | ...... | .... | ..... | 827.50 |
| 21 | ...... | .... | ..... | 922.50 |
| 22 | ...... | .... | ..... | 1017.50 |
| 23 | w. 5 | 27.5 | 122.5 | 1112.50 |
| 24 | ...... | .... | ..... | 1235.00 |
| 25 | ...... | .... | ..... | 1357.50 |
| 26 | ...... | .... | ..... | 1480.00 |
| 27 | ...... | .... | ..... | 1602.50 |
| 28 | ...... | .... | ..... | 1725.00 |
| 29 | ...... | .... | ..... | 1847.50 |
| 30 | ...... | .... | ..... | 1970.00 |
| 31 | ...... | .... | ..... | 2092.50 |
| 32 | w. 6 | 17.5 | 140.0 | 2215.00 |
| 33 | ...... | .... | ..... | 2355.00 |
| 34 | ...... | .... | ..... | 2495.00 |
| 35 | ...... | .... | ..... | 2635.00 |
| 36 | ...... | .... | ..... | 2775.00 |
| 37 | w. 7 | 17.5 | 157.5 | 2915.00 |
| 38 | ...... | .... | ..... | 3072.50 |
| 39 | ...... | .... | ..... | 3230.00 |
| 40 | ...... | .... | ..... | 3387.50 |
| 41 | ...... | .... | ..... | 3545.00 |
| 42 | ...... | .... | ..... | 3702.50 |
| 43 | w. 8 | 17.5 | 175.0 | 3860.00 |
| 44 | ...... | .... | ..... | 4035.00 |
| 45 | ...... | .... | ..... | 4210.00 |
| 46 | ...... | .... | ..... | 4385.00 |
| 47 | ...... | .... | ..... | 4560.00 |
| 48 | w. 9 | 17.5 | 192.5 | 4735.00 |
| 49 | ...... | .... | ..... | 4927.50 |
| 50 | ...... | .... | ..... | 5120.00 |

COMMON STANDARD 50′–100′

| Length | Wheel | Load | Load Sums | Moment Sums |
|---|---|---|---|---|
| 50 | ...... | .... | ..... | 5120.00 |
| 51 | ...... | .... | ..... | 5312.50 |
| 52 | ...... | .... | ..... | 5505.00 |
| 53 | ...... | .... | ..... | 5697.50 |
| 54 | ...... | .... | ..... | 5890.00 |
| 55 | ...... | .... | ..... | 6082.50 |
| 56 | w. 10 | 12.5 | 205.0 | 6275.00 |
| 57 | ...... | .... | ..... | 6480.00 |
| 58 | ...... | .... | ..... | 6685.00 |
| 59 | ...... | .... | ..... | 6890.00 |
| 60 | ...... | .... | ..... | 7095.00 |
| 61 | ...... | .... | ..... | 7300.00 |
| 62 | ...... | .... | ..... | 7505.00 |
| 63 | ...... | .... | ..... | 7710.00 |
| 64 | w. 11 | 27.5 | 232.5 | 7915.00 |
| 65 | ...... | .... | ..... | 8147.50 |
| 66 | ...... | .... | ..... | 8380.00 |
| 67 | ...... | .... | ..... | 8612.50 |
| 68 | ...... | .... | ..... | 8845.00 |
| 69 | w. 12 | 27.5 | 260.0 | 9077.50 |
| 70 | ...... | .... | ..... | 9337.50 |
| 71 | ...... | .... | ..... | 9597.50 |
| 72 | ...... | .... | ..... | 9857.50 |
| 73 | ...... | .... | ..... | 10117.50 |
| 74 | w. 13 | 27.5 | 287.5 | 10377.50 |
| 75 | ...... | .... | ..... | 10665.00 |
| 76 | ...... | .... | ..... | 10952.50 |
| 77 | ...... | .... | ..... | 11240.00 |
| 78 | ...... | .... | ..... | 11527.50 |
| 79 | w. 14 | 27.5 | 315.0 | 11815.00 |
| 80 | ...... | .... | ..... | 12130.00 |
| 81 | ...... | .... | ..... | 12445.00 |
| 82 | ...... | .... | ..... | 12760.00 |
| 83 | ...... | .... | ..... | 13075.00 |
| 84 | ...... | .... | ..... | 13390.00 |
| 85 | ...... | .... | ..... | 13705.00 |
| 86 | ...... | .... | ..... | 14020.00 |
| 87 | ...... | .... | ..... | 14335.00 |
| 88 | w. 15 | 17.5 | 332.5 | 14650.00 |
| 89 | ...... | .... | ..... | 14982.50 |
| 90 | ...... | .... | ..... | 15315.00 |
| 91 | ...... | .... | ..... | 15647.50 |
| 92 | ...... | .... | ..... | 15980.00 |
| 93 | w. 16 | 17.5 | 350.0 | 16312.50 |
| 94 | ...... | .... | ..... | 16662.50 |
| 95 | ...... | .... | ..... | 17012.50 |
| 96 | ...... | .... | ..... | 17362.50 |
| 97 | ...... | .... | ..... | 17712.50 |
| 98 | ...... | .... | ..... | 18062.50 |
| 99 | w. 17 | 17.5 | 367.5 | 18412.50 |
| 100 | ...... | .... | ..... | 18780.00 |

| Common Standard 100′–150′ | | | | | Common Standard 150′–200′ | | | |
|---|---|---|---|---|---|---|---|---|
| Length | Wheel | Load | Load Sums | Moment Sums | Length | Load | Load Sums | Moment Sums |
| 100 | ...... | .... | ..... | 18780.00 | 150 | | 487.5 | 40061.25 |
| 101 | ...... | .... | ..... | 19147.50 | 151 | | 490.0 | 40550.00 |
| 102 | ...... | .... | ..... | 19515.00 | 152 | | 492.5 | 41041.25 |
| 103 | ...... | .... | ..... | 19882.50 | 153 | | 495.0 | 41535.00 |
| 104 | w. 18 | 17.5 | 385.0 | 20250.00 | 154 | | 497.5 | 42031.25 |
| 105 | ...... | .... | ..... | 20635.00 | 155 | | 500.0 | 42530.00 |
| 106 | ...... | .... | ..... | 21020.00 | 156 | | 502.5 | 43031.25 |
| 107 | ...... | .... | ..... | 21405.00 | 157 | | 505.0 | 43535.00 |
| 108 | ...... | .... | ..... | 21790.00 | 158 | | 507.5 | 44041.25 |
| 109 | | | 385.0 | 22175.00 | 159 | | 510.0 | 44550.00 |
| 110 | | Uniform Load = 2,500 pounds per foot | 387.5 | 22561.25 | 160 | Uniform Load = 2,500 pounds per foot | 512.5 | 45061.25 |
| 111 | | | 390.0 | 22950.00 | 161 | | 515.0 | 45575.00 |
| 112 | | | 392.5 | 23341.25 | 162 | | 517.5 | 46091.25 |
| 113 | | | 395.0 | 23735.00 | 163 | | 520.0 | 46610.00 |
| 114 | | | 397.5 | 24131.25 | 164 | | 522.5 | 47131.25 |
| 115 | | | 400.0 | 24530.00 | 165 | | 525.0 | 47655.00 |
| 116 | | | 402.5 | 24931.25 | 166 | | 527.5 | 48181.25 |
| 117 | | | 405.0 | 25335.00 | 167 | | 530.0 | 48710.00 |
| 118 | | | 407.5 | 25741.25 | 168 | | 532.5 | 49241.25 |
| 119 | | | 410.0 | 26150.00 | 169 | | 535.0 | 49775.00 |
| 120 | | | 412.5 | 26561.25 | 170 | | 537.5 | 50311.25 |
| 121 | | | 415.0 | 26975.00 | 171 | | 540.0 | 50850.00 |
| 122 | | | 417.5 | 27391.25 | 172 | | 542.5 | 51391.25 |
| 123 | | | 420.0 | 27810.00 | 173 | | 545.0 | 51935.00 |
| 124 | | | 422.5 | 28231.25 | 174 | | 547.5 | 52481.25 |
| 125 | | | 425.0 | 28655.00 | 175 | | 550.0 | 53030.00 |
| 126 | | | 427.5 | 29081.25 | 176 | | 552.5 | 53581.25 |
| 127 | | | 430.0 | 29510.00 | 177 | | 555.0 | 54135.00 |
| 128 | | | 432.5 | 29941.25 | 178 | | 557.5 | 54691.25 |
| 129 | | | 435.0 | 30375.00 | 179 | | 560.0 | 55250.00 |
| 130 | | | 437.5 | 30811.25 | 180 | | 562.5 | 55811.25 |
| 131 | | | 440.0 | 31250.00 | 181 | | 565.0 | 56375.00 |
| 132 | | | 442.5 | 31691.25 | 182 | | 567.5 | 56941.25 |
| 133 | | | 445.0 | 32135.00 | 183 | | 570.0 | 57510.00 |
| 134 | | | 447.5 | 32581.25 | 184 | | 572.5 | 58081.25 |
| 135 | | | 450.0 | 33030.00 | 185 | | 575.0 | 58655.00 |
| 136 | | | 452.5 | 33481.25 | 186 | | 577.5 | 59231.25 |
| 137 | | | 455.0 | 33935.00 | 187 | | 580.0 | 59810.00 |
| 138 | | | 457.5 | 34391.25 | 188 | | 582.5 | 60391.25 |
| 139 | | | 460.0 | 34850.00 | 189 | | 585.0 | 60975.00 |
| 140 | | | 462.5 | 35311.25 | 190 | | 587.5 | 61561.25 |
| 141 | | | 465.0 | 35775.00 | 191 | | 590.0 | 62150.00 |
| 142 | | | 467.5 | 36241.25 | 192 | | 592.5 | 62741.25 |
| 143 | | | 470.0 | 36710.00 | 193 | | 595.0 | 63335.00 |
| 144 | | | 472.5 | 37181.25 | 194 | | 597.5 | 63931.25 |
| 145 | | | 475.0 | 37655.00 | 195 | | 600.0 | 64530.00 |
| 146 | | | 477.5 | 38131.25 | 196 | | 602.5 | 65131.25 |
| 147 | | | 480.0 | 38610.00 | 197 | | 605.0 | 65735.00 |
| 148 | | | 482.5 | 39091.25 | 198 | | 607.5 | 66341.25 |
| 149 | | | 485.0 | 39575.00 | 199 | | 610.0 | 66950.00 |
| 150 | | | 487.5 | 40061.25 | 200 | | 612.5 | 67561.25 |

Common Standard 200′–250′

| Length | Load | Load Sums | Moment Sums |
|---|---|---|---|
| 200 | Uniform Load = 2,500 pounds per foot | 612.5 | 67561.25 |
| 201 | | 615.0 | 68175.00 |
| 202 | | 617.5 | 68791.25 |
| 203 | | 620.0 | 69410.00 |
| 204 | | 622.5 | 70031.25 |
| 205 | | 625.0 | 70655.00 |
| 206 | | 627.5 | 71281.25 |
| 207 | | 630.0 | 71910.00 |
| 208 | | 632.5 | 72541.25 |
| 209 | | 635.0 | 73175.00 |
| 210 | | 637.5 | 73811.25 |
| 211 | | 640.0 | 74450.00 |
| 212 | | 642.5 | 75091.25 |
| 213 | | 645.0 | 75735.00 |
| 214 | | 647.5 | 76381.25 |
| 215 | | 650.0 | 77030.00 |
| 216 | | 652.5 | 77681.25 |
| 217 | | 655.0 | 78335.00 |
| 218 | | 657.5 | 78991.25 |
| 219 | | 660.0 | 79650.00 |
| 220 | | 662.5 | 80311.25 |
| 221 | | 665.0 | 80975.00 |
| 222 | | 667.5 | 81641.25 |
| 223 | | 670.0 | 82310.00 |
| 224 | | 672.5 | 82981.25 |
| 225 | | 675.0 | 83655.00 |
| 226 | | 677.5 | 84331.25 |
| 227 | | 680.0 | 85010.00 |
| 228 | | 682.5 | 85691.25 |
| 229 | | 685.0 | 86375.00 |
| 230 | | 687.5 | 87061.25 |
| 231 | | 690.0 | 87750.00 |
| 232 | | 692.5 | 88441.25 |
| 233 | | 695.0 | 89135.00 |
| 234 | | 697.5 | 89831.25 |
| 235 | | 700.0 | 90530.00 |
| 236 | | 702.5 | 91231.25 |
| 237 | | 705.0 | 91935.00 |
| 238 | | 707.5 | 92641.25 |
| 239 | | 710.0 | 93350.00 |
| 240 | | 712.5 | 94061.25 |
| 241 | | 715.0 | 94775.00 |
| 242 | | 717.5 | 95491.25 |
| 243 | | 720.0 | 96210.00 |
| 244 | | 722.5 | 96931.25 |
| 245 | | 725.0 | 97655.00 |
| 246 | | 727.5 | 98381.25 |
| 247 | | 730.0 | 99110.00 |
| 248 | | 732.5 | 99841.25 |
| 249 | | 735.0 | 100575.00 |
| 250 | | 737.5 | 101311.25 |

Common Standard 250′–300′

| Length | Load | Load Sums | Moment Sums |
|---|---|---|---|
| 250 | Uniform Load = 2,500 pounds per foot | 737.5 | 101311.25 |
| 251 | | 740.0 | 102050.00 |
| 252 | | 742.5 | 102791.25 |
| 253 | | 745.0 | 103535.00 |
| 254 | | 747.5 | 104281.25 |
| 255 | | 750.0 | 105030.00 |
| 256 | | 752.5 | 105781.25 |
| 257 | | 755.0 | 106535.00 |
| 258 | | 757.5 | 107291.25 |
| 259 | | 760.0 | 108050.00 |
| 260 | | 762.5 | 108811.25 |
| 261 | | 765.0 | 109575.00 |
| 262 | | 767.5 | 110341.25 |
| 263 | | 770.0 | 111110.00 |
| 264 | | 772.5 | 111881.25 |
| 265 | | 775.0 | 112655.00 |
| 266 | | 777.5 | 113431.25 |
| 267 | | 780.0 | 114210.00 |
| 268 | | 782.5 | 114991.25 |
| 269 | | 785.0 | 115775.00 |
| 270 | | 787.5 | 116561.25 |
| 271 | | 790.0 | 117350.00 |
| 272 | | 792.5 | 118141.25 |
| 273 | | 795.0 | 118935.00 |
| 274 | | 797.5 | 119731.25 |
| 275 | | 800.0 | 120530.00 |
| 276 | | 802.5 | 121331.25 |
| 277 | | 805.0 | 122135.00 |
| 278 | | 807.5 | 122941.25 |
| 279 | | 810.0 | 123750.00 |
| 280 | | 812.5 | 124561.25 |
| 281 | | 815.0 | 125375.00 |
| 282 | | 817.5 | 126191.25 |
| 283 | | 820.0 | 127010.00 |
| 284 | | 822.5 | 127831.25 |
| 285 | | 825.0 | 128655.00 |
| 286 | | 827.5 | 129481.25 |
| 287 | | 830.0 | 130310.00 |
| 288 | | 832.5 | 131141.25 |
| 289 | | 835.0 | 131975.00 |
| 290 | | 837.5 | 132811.25 |
| 291 | | 840.0 | 133650.00 |
| 292 | | 842.5 | 134491.25 |
| 293 | | 845.0 | 135335.00 |
| 294 | | 847.5 | 136181.25 |
| 295 | | 850.0 | 137030.00 |
| 296 | | 852.5 | 137881.25 |
| 297 | | 855.0 | 138735.00 |
| 298 | | 857.5 | 139591.25 |
| 299 | | 860.0 | 140450.00 |
| 300 | | 862.5 | 141311.25 |

COMMON STANDARD 300′–350′ | COMMON STANDARD 350′–400′

| Length | Load | Load Sums | Moment Sums | Length | Load | Load Sums | Moment Sums |
|---|---|---|---|---|---|---|---|
| 300 | Uniform Load=2,500 pounds per foot | 862.5 | 141311.25 | 350 | Uniform Load=2,500 pounds per foot | 987.50 | 187561.25 |
| 301 | | 865.0 | 142175.00 | 351 | | 990.00 | 188550.00 |
| 302 | | 867.5 | 143041.25 | 352 | | 992.50 | 189541.25 |
| 303 | | 870.0 | 143910.00 | 353 | | 995.00 | 190535.00 |
| 304 | | 872.5 | 144781.25 | 354 | | 997.50 | 191531.25 |
| 305 | | 875.0 | 145655.00 | 355 | | 1000.00 | 192530.00 |
| 306 | | 877.5 | 146531.25 | 356 | | 1002.50 | 193531.25 |
| 307 | | 880.0 | 147410.00 | 357 | | 1005.00 | 194535.00 |
| 308 | | 882.5 | 148291.25 | 358 | | 1007.50 | 195541.25 |
| 309 | | 885.0 | 149175.00 | 359 | | 1010.00 | 196550.00 |
| 310 | | 887.5 | 150061.25 | 360 | | 1012.50 | 197561.25 |
| 311 | | 890.0 | 150950.00 | 361 | | 1015.00 | 198575.00 |
| 312 | | 892.5 | 151841.25 | 362 | | 1017.50 | 199591.25 |
| 313 | | 895.0 | 152735.00 | 363 | | 1020.00 | 200610.00 |
| 314 | | 897.5 | 153631.25 | 364 | | 1022.50 | 201631.25 |
| 315 | | 900.0 | 154530.00 | 365 | | 1025.00 | 202655.00 |
| 316 | | 902.5 | 155431.25 | 366 | | 1027.50 | 203681.25 |
| 317 | | 905.0 | 156335.00 | 367 | | 1030.00 | 204710.00 |
| 318 | | 907.5 | 157241.25 | 368 | | 1032.50 | 205741.25 |
| 319 | | 910.0 | 158150.00 | 369 | | 1035.00 | 206775.00 |
| 320 | | 912.5 | 159061.25 | 370 | | 1037.50 | 207811.25 |
| 321 | | 915.0 | 159975.00 | 371 | | 1040.00 | 208850.00 |
| 322 | | 917.5 | 160891.25 | 372 | | 1042.50 | 209891.25 |
| 323 | | 920.0 | 161810.00 | 373 | | 1045.00 | 210935.00 |
| 324 | | 922.5 | 162731.25 | 374 | | 1047.50 | 211981.25 |
| 325 | | 925.0 | 163655.00 | 375 | | 1050.00 | 213030.00 |
| 326 | | 927.5 | 164581.25 | 376 | | 1052.50 | 214081.25 |
| 327 | | 930.0 | 165510.00 | 377 | | 1055.00 | 215135.00 |
| 328 | | 932.5 | 166441.25 | 378 | | 1057.50 | 216191.25 |
| 329 | | 935.0 | 167375.00 | 379 | | 1060.00 | 217250.00 |
| 330 | | 937.5 | 168311.25 | 380 | | 1062.50 | 218311.25 |
| 331 | | 940.0 | 169250.00 | 381 | | 1065.00 | 219375.00 |
| 332 | | 942.5 | 170191.25 | 382 | | 1067.50 | 220441.25 |
| 333 | | 945.0 | 171135.00 | 383 | | 1070.00 | 221510.00 |
| 334 | | 947.5 | 172081.25 | 384 | | 1072.50 | 222581.25 |
| 335 | | 950.0 | 173030.00 | 385 | | 1075.00 | 223655.00 |
| 336 | | 952.5 | 173981.25 | 386 | | 1077.50 | 224731.25 |
| 337 | | 955.0 | 174935.00 | 387 | | 1080.00 | 225810.00 |
| 338 | | 957.5 | 175891.25 | 388 | | 1082.50 | 226891.25 |
| 339 | | 960.0 | 176850.00 | 389 | | 1085.00 | 227975.00 |
| 340 | | 962.5 | 177811.25 | 390 | | 1087.50 | 229061.25 |
| 341 | | 965.0 | 178775.00 | 391 | | 1090.00 | 230150.00 |
| 342 | | 967.5 | 179741.25 | 392 | | 1092.50 | 231241.25 |
| 343 | | 970.0 | 180710.00 | 393 | | 1095.00 | 232335.00 |
| 344 | | 972.5 | 181681.25 | 394 | | 1097.50 | 233431.25 |
| 345 | | 975.0 | 182655.00 | 395 | | 1100.00 | 234530.00 |
| 346 | | 977.5 | 183631.25 | 396 | | 1102.50 | 235631.25 |
| 347 | | 980.0 | 184610.00 | 397 | | 1105.00 | 236735.00 |
| 348 | | 982.5 | 185591.25 | 398 | | 1107.50 | 237841.25 |
| 349 | | 985.0 | 186575.00 | 399 | | 1110.00 | 238950.00 |
| 350 | | 987.5 | 187561.25 | 400 | | 1112.50 | 240061.25 |

| Lackawanna 0′–50′ | | | | | Lackawanna 50′–100′ | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Length | Wheel | Load | Load Sums | Moment Sums | Length | Wheel | Load | Load Sums | Moment Sums |
| 0 | w. 1 | 11 | 11.00 | 00.000 | 50 | ...... | .. | ...... | 4744.000 |
| 1 | ...... | .. | ...... | 11.000 | 51 | ...... | .. | ...... | 4911.000 |
| 2 | ...... | .. | ...... | 22.000 | 52 | ...... | .. | ...... | 5078.000 |
| 3 | ...... | .. | ...... | 33.000 | 53 | ...... | .. | ...... | 5245.000 |
| 4 | ...... | .. | ...... | 44.000 | 54 | w. 10 | 11 | 178.00 | 5412.000 |
| 5 | ...... | .. | ...... | 55.000 | 55 | ...... | .. | ...... | 5590.000 |
| 6 | ...... | .. | ...... | 66.000 | 56 | ...... | .. | ...... | 5768.000 |
| 7 | w. 2 | 25 | 36.00 | 77.000 | 57 | ...... | .. | ...... | 5946.000 |
| 8 | ...... | .. | ...... | 113.000 | 58 | ...... | .. | ...... | 6124.000 |
| 9 | ...... | .. | ...... | 149.000 | 59 | ...... | .. | ...... | 6302.000 |
| 10 | ...... | .. | ...... | 185.000 | 60 | ...... | .. | ...... | 6480.000 |
| 11 | ...... | .. | ...... | 221.000 | 61 | w. 11 | 25 | 203.00 | 6658.000 |
| 12 | w. 3 | 25 | 61.00 | 257.000 | 62 | ...... | .. | ...... | 6861.000 |
| 13 | ...... | .. | ...... | 318.000 | 63 | ...... | .. | ...... | 7064.000 |
| 14 | ...... | .. | ...... | 379.000 | 64 | ...... | .. | ...... | 7267.000 |
| 15 | ...... | .. | ...... | 440.000 | 65 | ...... | .. | ...... | 7470.000 |
| 16 | ...... | .. | ...... | 501.000 | 66 | w. 12 | 25 | 228.00 | 7673.000 |
| 17 | w. 4 | 25 | 86.00 | 562.000 | 67 | ...... | .. | ...... | 7901.000 |
| 18 | ...... | .. | ...... | 648.000 | 68 | ...... | .. | ...... | 8129.000 |
| 19 | ...... | .. | ...... | 734.000 | 69 | ...... | .. | ...... | 8357.000 |
| 20 | ...... | .. | ...... | 820.000 | 70 | ...... | .. | ...... | 8585.000 |
| 21 | ...... | .. | ...... | 906.000 | 71 | w. 13 | 25 | 253.00 | 8813.000 |
| 22 | w. 5 | 25 | 111.00 | 992.000 | 72 | ...... | .. | ...... | 9066.000 |
| 23 | ...... | .. | ...... | 1103.000 | 73 | ...... | .. | ...... | 9319.000 |
| 24 | ...... | .. | ...... | 1214.000 | 74 | ...... | .. | ...... | 9572.000 |
| 25 | ...... | .. | ...... | 1325.000 | 75 | ...... | .. | ...... | 9825.000 |
| 26 | ...... | .. | ...... | 1436.000 | 76 | w. 14 | 25 | 278.00 | 10078.000 |
| 27 | ...... | .. | ...... | 1547.000 | 77 | ...... | .. | ...... | 10356.000 |
| 28 | ...... | .. | ...... | 1658.000 | 78 | ...... | .. | ...... | 10634.000 |
| 29 | ...... | .. | ...... | 1769.000 | 79 | ...... | .. | ...... | 10912.000 |
| 30 | ...... | .. | ...... | 1880.000 | 80 | ...... | .. | ...... | 11190.000 |
| 31 | w. 6 | 14 | 125.00 | 1991.000 | 81 | ...... | .. | ...... | 11468.000 |
| 32 | ...... | .. | ...... | 2116.000 | 82 | ...... | .. | ...... | 11746.000 |
| 33 | ...... | .. | ...... | 2241.000 | 83 | ...... | .. | ...... | 12024.000 |
| 34 | ...... | .. | ...... | 2366.000 | 84 | ...... | .. | ...... | 12302.000 |
| 35 | ...... | .. | ...... | 2491.000 | 85 | w. 15 | 14 | 292.00 | 12580.000 |
| 36 | w. 7 | 14 | 139.00 | 2616.000 | 86 | ...... | .. | ...... | 12872.000 |
| 37 | ...... | .. | ...... | 2755.000 | 87 | ...... | .. | ...... | 13146.000 |
| 38 | ...... | .. | ...... | 2894.000 | 88 | ...... | .. | ...... | 13456.000 |
| 39 | ...... | .. | ...... | 3033.000 | 89 | ...... | .. | ...... | 13748.000 |
| 40 | ...... | .. | ...... | 3172.000 | 90 | w. 16 | 14 | 306.00 | 14040.000 |
| 41 | w. 8 | 14 | 153.00 | 3311.000 | 91 | ...... | .. | ...... | 14346.000 |
| 42 | ...... | .. | ...... | 3464.000 | 92 | ...... | .. | ...... | 14652.000 |
| 43 | ...... | .. | ...... | 3617.000 | 93 | ...... | .. | ...... | 14958.000 |
| 44 | ...... | .. | ...... | 3770.000 | 94 | ...... | .. | ...... | 15264.000 |
| 45 | ...... | .. | ...... | 3923.000 | 95 | w. 17 | 14 | 320.00 | 15570.000 |
| 46 | w. 9 | 14 | 167.00 | 4076.000 | 96 | ...... | .. | ...... | 15890.000 |
| 47 | ...... | .. | ...... | 4243.000 | 97 | ...... | .. | ...... | 16210.000 |
| 48 | ...... | .. | ...... | 4410.000 | 98 | ...... | .. | ...... | 16530.000 |
| 49 | ...... | .. | ...... | 4577.000 | 99 | ...... | .. | ...... | 16850.000 |
| 50 | ...... | .. | ...... | 4744.000 | 100 | w. 18 | 14 | 334.00 | 17170.000 |

LACKAWANNA 100′–150′

| Length | Wheel | Load | Load Sums | Moment Sums |
|---|---|---|---|---|
| 100 | w. 18 | 14 | 334.00 | 17170.000 |
| 101 | ...... | .. | ...... | 17504.000 |
| 102 | ...... | .. | ...... | 17838.000 |
| 103 | ...... | .. | ...... | 18172.000 |
| 104 | | Uniform Load = 2,250 pounds per foot | 334.00 | 18506.000 |
| 105 | | | 336.25 | 18841.125 |
| 106 | | | 338.50 | 19178.500 |
| 107 | | | 340.75 | 19518.125 |
| 108 | | | 343.00 | 19860.000 |
| 109 | | | 345.25 | 20204.125 |
| 110 | | | 347.50 | 20550.500 |
| 111 | | | 349.75 | 20899.125 |
| 112 | | | 352.00 | 21250.000 |
| 113 | | | 354.25 | 21603.125 |
| 114 | | | 356.50 | 21958.500 |
| 115 | | | 358.75 | 22316.125 |
| 116 | | | 361.00 | 22676.000 |
| 117 | | | 363.25 | 23038.125 |
| 118 | | | 365.50 | 23402.500 |
| 119 | | | 367.75 | 23769.125 |
| 120 | | | 370.00 | 24138.000 |
| 121 | | | 372.25 | 24509.125 |
| 122 | | | 374.50 | 24882.500 |
| 123 | | | 376.75 | 25258.125 |
| 124 | | | 379.00 | 25636.000 |
| 125 | | | 381.25 | 26016.125 |
| 126 | | | 383.50 | 26398.500 |
| 127 | | | 385.75 | 26783.125 |
| 128 | | | 388.00 | 27170.000 |
| 129 | | | 390.25 | 27559.125 |
| 130 | | | 392.50 | 27950.500 |
| 131 | | | 394.75 | 28344.125 |
| 132 | | | 397.00 | 28740.000 |
| 133 | | | 399.25 | 29138.125 |
| 134 | | | 401.50 | 29538.500 |
| 135 | | | 403.75 | 29941.125 |
| 136 | | | 406.00 | 30346.000 |
| 137 | | | 408.25 | 30753.125 |
| 138 | | | 410.50 | 31162.500 |
| 139 | | | 412.75 | 31574.125 |
| 140 | | | 415.00 | 31988.000 |
| 141 | | | 417.25 | 32404.125 |
| 142 | | | 419.50 | 32882.500 |
| 143 | | | 421.75 | 33243.125 |
| 144 | | | 424.00 | 33666.000 |
| 145 | | | 426.25 | 34091.125 |
| 146 | | | 428.50 | 34518.500 |
| 147 | | | 430.75 | 34948.125 |
| 148 | | | 433.00 | 35380.000 |
| 149 | | | 435.25 | 35814.125 |
| 150 | | | 437.50 | 36250.500 |

LACKAWANNA 150′–200′

| Length | Load | Load Sums | Moment Sums |
|---|---|---|---|
| 150 | Uniform Load = 2,250 pounds per foot | 437.50 | 36250.500 |
| 151 | | 439.75 | 36689.125 |
| 152 | | 442.00 | 37130.000 |
| 153 | | 444.25 | 37573.125 |
| 154 | | 446.50 | 38018.500 |
| 155 | | 448.75 | 38466.125 |
| 156 | | 451.00 | 38916.000 |
| 157 | | 453.25 | 39368.125 |
| 158 | | 455.50 | 39822.500 |
| 159 | | 457.75 | 40279.125 |
| 160 | | 460.00 | 40738.000 |
| 161 | | 462.25 | 41199.125 |
| 162 | | 464.50 | 41662.500 |
| 163 | | 466.75 | 42128.125 |
| 164 | | 469.00 | 42596.000 |
| 165 | | 471.25 | 43066.125 |
| 166 | | 473.50 | 43538.500 |
| 167 | | 475.75 | 44013.125 |
| 168 | | 478.00 | 44490.000 |
| 169 | | 480.25 | 44969.125 |
| 170 | | 482.50 | 45450.500 |
| 171 | | 484.75 | 45934.125 |
| 172 | | 487.00 | 46420.000 |
| 173 | | 489.25 | 46908.125 |
| 174 | | 491.50 | 47398.500 |
| 175 | | 493.75 | 47891.125 |
| 176 | | 496.00 | 48386.000 |
| 177 | | 498.25 | 48883.125 |
| 178 | | 500.50 | 49382.500 |
| 179 | | 502.75 | 49884.125 |
| 180 | | 505.00 | 50338.000 |
| 181 | | 507.25 | 50894.125 |
| 182 | | 509.50 | 51402.500 |
| 183 | | 511.75 | 51913.125 |
| 184 | | 514.00 | 52426.000 |
| 185 | | 516.25 | 52941.125 |
| 186 | | 518.50 | 53458.500 |
| 187 | | 520.75 | 53978.125 |
| 188 | | 523.00 | 54500.000 |
| 189 | | 525.25 | 55024.125 |
| 190 | | 527.50 | 55550.500 |
| 191 | | 529.75 | 56079.125 |
| 192 | | 532.00 | 56610.000 |
| 193 | | 534.25 | 57143.125 |
| 194 | | 536.50 | 57678.500 |
| 195 | | 538.75 | 58216.125 |
| 196 | | 541.00 | 58756.000 |
| 197 | | 543.25 | 59298.125 |
| 198 | | 545.50 | 59842.500 |
| 199 | | 547.75 | 60389.125 |
| 200 | | 550.00 | 60938.000 |

LACKAWANNA 200′–250′

| Length | Load | Load Sums | Moment Sums |
|---|---|---|---|
| 200 | Uniform Load = 2,250 pounds per foot | 550.00 | 60938.000 |
| 201 | | 552.25 | 61489.125 |
| 202 | | 554.50 | 62042.500 |
| 203 | | 556.75 | 62598.125 |
| 204 | | 559.00 | 63156.000 |
| 205 | | 561.25 | 63716.125 |
| 206 | | 563.50 | 64278.500 |
| 207 | | 565.75 | 64843.125 |
| 208 | | 568.00 | 65410.000 |
| 209 | | 570.25 | 65979.125 |
| 210 | | 572.50 | 66550.500 |
| 211 | | 574.75 | 67124.125 |
| 212 | | 577.00 | 67700.000 |
| 213 | | 579.25 | 68278.125 |
| 214 | | 581.50 | 68858.500 |
| 215 | | 583.75 | 69441.125 |
| 216 | | 586.00 | 70026.000 |
| 217 | | 588.25 | 70613.125 |
| 218 | | 590.50 | 71202.500 |
| 219 | | 592.75 | 71794.125 |
| 220 | | 595.00 | 72388.000 |
| 221 | | 597.25 | 72984.125 |
| 222 | | 599.50 | 73582.500 |
| 223 | | 601.75 | 74183.125 |
| 224 | | 604.00 | 74786.000 |
| 225 | | 606.25 | 75391.125 |
| 226 | | 608.50 | 75998.500 |
| 227 | | 610.75 | 76608.125 |
| 228 | | 613.00 | 77220.000 |
| 229 | | 615.25 | 77834.125 |
| 230 | | 617.50 | 78450.500 |
| 231 | | 619.75 | 79069.125 |
| 232 | | 622.00 | 79690.000 |
| 233 | | 624.25 | 80313.125 |
| 234 | | 626.50 | 80938.500 |
| 235 | | 628.75 | 81566.125 |
| 236 | | 631.00 | 82196.000 |
| 237 | | 633.25 | 82828.125 |
| 238 | | 635.50 | 83462.500 |
| 239 | | 637.75 | 84099.125 |
| 240 | | 640.00 | 84738.000 |
| 241 | | 642.25 | 85379.125 |
| 242 | | 644.50 | 86022.500 |
| 243 | | 646.75 | 86668.125 |
| 244 | | 649.00 | 87316.000 |
| 245 | | 651.25 | 87966.125 |
| 246 | | 653.50 | 88618.500 |
| 247 | | 655.75 | 89273.125 |
| 248 | | 658.00 | 89930.000 |
| 249 | | 660.25 | 90589.125 |
| 250 | | 662.50 | 91250.500 |

LACKAWANNA 250′–300′

| Length | Load | Load Sums | Moment Sums |
|---|---|---|---|
| 250 | Uniform Load = 2,250 pounds per foot | 662.50 | 91250.500 |
| 251 | | 664.75 | 91914.125 |
| 252 | | 667.00 | 92580.000 |
| 253 | | 669.25 | 93248.125 |
| 254 | | 671.50 | 93918.500 |
| 255 | | 673.75 | 94591.125 |
| 256 | | 676.00 | 95266.000 |
| 257 | | 678.25 | 95943.125 |
| 258 | | 680.50 | 96622.500 |
| 259 | | 682.75 | 97304.125 |
| 260 | | 685.00 | 97988.000 |
| 261 | | 687.25 | 98674.125 |
| 262 | | 689.50 | 99362.500 |
| 263 | | 691.75 | 100053.125 |
| 264 | | 694.00 | 100746.000 |
| 265 | | 696.25 | 101441.125 |
| 266 | | 698.50 | 102138.500 |
| 267 | | 700.75 | 102838.125 |
| 268 | | 703.00 | 103540.000 |
| 269 | | 705.25 | 104244.125 |
| 270 | | 707.50 | 105950.500 |
| 271 | | 709.75 | 105659.125 |
| 272 | | 712.00 | 106370.000 |
| 273 | | 714.25 | 107083.125 |
| 274 | | 716.50 | 107798.500 |
| 275 | | 718.75 | 108516.125 |
| 276 | | 721.00 | 109236.000 |
| 277 | | 723.25 | 109958.125 |
| 278 | | 725.50 | 110682.500 |
| 279 | | 727.75 | 111409.125 |
| 280 | | 730.00 | 112138.000 |
| 281 | | 732.25 | 112869.125 |
| 282 | | 734.50 | 113602.500 |
| 283 | | 736.75 | 114338.125 |
| 284 | | 739.00 | 115076.000 |
| 285 | | 741.25 | 115816.125 |
| 286 | | 743.50 | 116558.500 |
| 287 | | 745.75 | 117303.125 |
| 288 | | 748.00 | 118050.000 |
| 289 | | 750.25 | 118799.125 |
| 290 | | 752.50 | 119550.500 |
| 291 | | 754.75 | 120304.125 |
| 292 | | 757.00 | 121060.000 |
| 293 | | 759.25 | 121818.125 |
| 294 | | 761.50 | 122578.500 |
| 295 | | 763.75 | 123341.125 |
| 296 | | 766.00 | 124106.000 |
| 297 | | 768.25 | 124873.125 |
| 298 | | 770.50 | 125642.500 |
| 299 | | 772.75 | 126414.125 |
| 300 | | 775.00 | 127188.000 |

LACKAWANNA 300′–350′

| Length | Load | Load Sums | Moment Sums |
|---|---|---|---|
| 300 | Uniform Load = 2,250 pounds per foot | 775.00 | 127188.000 |
| 301 | | 777.25 | 127964.125 |
| 302 | | 779.50 | 128742.500 |
| 303 | | 781.75 | 129523.125 |
| 304 | | 784.00 | 130306.000 |
| 305 | | 786.25 | 131091.125 |
| 306 | | 788.50 | 131878.500 |
| 307 | | 790.75 | 132668.125 |
| 308 | | 793.00 | 133460.000 |
| 309 | | 795.25 | 134254.125 |
| 310 | | 797.50 | 135050.500 |
| 311 | | 799.75 | 135849.125 |
| 312 | | 802.00 | 136650.000 |
| 313 | | 804.25 | 137453.125 |
| 314 | | 806.50 | 138258.500 |
| 315 | | 808.75 | 139066.125 |
| 316 | | 811.00 | 139876.000 |
| 317 | | 813.25 | 140688.125 |
| 318 | | 815.50 | 141502.500 |
| 319 | | 817.75 | 142319.125 |
| 320 | | 820.00 | 143138.000 |
| 321 | | 822.25 | 143959.125 |
| 322 | | 824.50 | 144782.500 |
| 323 | | 826.75 | 145608.125 |
| 324 | | 829.00 | 146436.000 |
| 325 | | 831.25 | 147266.125 |
| 326 | | 833.50 | 148098.500 |
| 327 | | 835.75 | 148933.125 |
| 328 | | 838.00 | 149770.000 |
| 329 | | 840.25 | 150609.125 |
| 330 | | 842.50 | 151450.500 |
| 331 | | 844.75 | 152294.125 |
| 332 | | 847.00 | 153140.000 |
| 333 | | 849.25 | 153988.125 |
| 334 | | 851.50 | 154838.500 |
| 335 | | 853.75 | 155691.125 |
| 336 | | 856.00 | 156546.000 |
| 337 | | 858.25 | 157403.125 |
| 338 | | 860.50 | 158262.500 |
| 339 | | 862.75 | 159124.125 |
| 340 | | 865.00 | 159988.000 |
| 341 | | 867.25 | 160854.125 |
| 342 | | 869.50 | 161722.500 |
| 343 | | 871.75 | 162593.125 |
| 344 | | 874.00 | 163466.000 |
| 345 | | 876.25 | 164341.125 |
| 346 | | 878.50 | 165218.500 |
| 347 | | 880.75 | 166098.125 |
| 348 | | 883.00 | 166980.000 |
| 349 | | 885.25 | 167864.125 |
| 350 | | 887.50 | 168750.500 |

LACKAWANNA 350′–400′

| Length | Load | Load Sums | Moment Sums |
|---|---|---|---|
| 350 | Uniform Load = 2,250 pounds per foot | 887.50 | 168750.500 |
| 351 | | 889.75 | 169639.125 |
| 352 | | 892.00 | 170530.000 |
| 353 | | 894.25 | 171423.125 |
| 354 | | 896.50 | 172318.500 |
| 355 | | 898.75 | 173216.125 |
| 356 | | 901.00 | 174116.000 |
| 357 | | 903.25 | 175018.125 |
| 358 | | 905.50 | 175922.500 |
| 359 | | 907.75 | 176829.125 |
| 360 | | 910.00 | 177738.000 |
| 361 | | 912.25 | 178649.125 |
| 362 | | 914.50 | 179562.500 |
| 363 | | 916.75 | 180478.125 |
| 364 | | 919.00 | 181396.000 |
| 365 | | 921.25 | 182316.125 |
| 366 | | 923.50 | 183238.500 |
| 367 | | 925.75 | 184163.125 |
| 368 | | 928.00 | 185090.000 |
| 369 | | 930.25 | 186019.125 |
| 370 | | 932.50 | 186950.500 |
| 371 | | 934.75 | 187884.125 |
| 372 | | 937.00 | 188820.000 |
| 373 | | 939.25 | 189758.125 |
| 374 | | 941.50 | 190698.500 |
| 375 | | 943.75 | 191641.125 |
| 376 | | 946.00 | 192586.000 |
| 377 | | 948.25 | 193533.125 |
| 378 | | 950.50 | 194482.500 |
| 379 | | 952.75 | 195434.125 |
| 380 | | 955.00 | 196388.000 |
| 381 | | 957.25 | 197344.125 |
| 382 | | 959.50 | 198302.500 |
| 383 | | 961.75 | 199263.125 |
| 384 | | 964.00 | 200226.000 |
| 385 | | 966.25 | 201191.125 |
| 386 | | 968.50 | 202158.500 |
| 387 | | 970.75 | 203128.125 |
| 388 | | 973.00 | 204100.000 |
| 389 | | 975.25 | 205074.125 |
| 390 | | 977.50 | 206050.500 |
| 391 | | 979.75 | 207029.125 |
| 392 | | 982.00 | 208010.000 |
| 393 | | 984.25 | 208993.125 |
| 394 | | 986.50 | 209978.500 |
| 395 | | 988.75 | 210966.125 |
| 396 | | 991.00 | 211956.000 |
| 397 | | 993.25 | 212948.125 |
| 398 | | 995.50 | 213942.500 |
| 399 | | 997.75 | 214939.125 |
| 400 | | 1000.00 | 215938.000 |

## TABLE 3

### Position of Cooper's Loadings for Maximum Stress

Shorter Segment $l_1$

Longer Segment $l_2$

| Segments | 5 | 10 | 15 | 20 | 25 | 30 | 35 | 40 | 45 | 50 | 55 | 60 | 65 | 70 | 75 | 80 | 85 | 90 | 95 | 100 | 110 | 120 | 130 | 140 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 300–260 | 2 | 2 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 7 | 7 | 8 | 9 | 10 | 10 | 11 | 11 | 12 | 12 | 13 | 14 | 15 | 17 | 18 |
| 250–200 | 2 | 2 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 7 | 8 | 8 | 9 | 10 | 11 | 11 | 12 | 12 | 12 | 13 | 14 | 15 | 17 | 18 |
| 190–150 | 2 | 2 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 7 | 8 | 9 | 9 | 11 | 11 | 12 | 12 | 12 | 12 | 13 | 14 | 15 | 17 | 18 |
| 140 | 2 | 3 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 12 | 12 | 12 | 12 | 13 | 14 | 15 | 17 | 18 |
| 130 | 2 | 3 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 12 | 12 | 12 | 12 | 13 | 14 | 15 | 17 | .. |
| 120 | 2 | 3 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 12 | 12 | 13 | 13 | 13 | 14 | 15 | .. | .. |
| 110 | 2 | 3 | 3 | 3 | 4 | 4 | 5 | 6 | 7 | 7 | 8 | 9 | 10 | 11 | 12 | 12 | 12 | 13 | 13 | 13 | 14 | .. | .. | .. |
| 100 | 2 | 3 | 3 | 3 | 4 | 5 | 5 | 6 | $\overline{14}$ | $\overline{14}$ | $\overline{14}$ | $\overline{13}$ | $\overline{13}$ | 11 | 12 | 12 | 12 | 13 | 13 | 13 | .. | .. | .. | .. |
| 95 | 2 | 3 | 3 | 4 | 4 | 5 | $\overline{13}$ | $\overline{13}$ | $\overline{13}$ | $\overline{13}$ | $\overline{13}$ | $\overline{13}$ | $\overline{13}$ | $\overline{13}$ | 12 | 12 | 12 | 13 | 13 | .. | .. | .. | .. | .. |
| 90 | 2 | 3 | 3 | 4 | 4 | 5 | $\overline{13}$ | $\overline{13}$ | $\overline{13}$ | $\overline{13}$ | $\overline{13}$ | $\overline{13}$ | $\overline{13}$ | $\overline{13}$ | $\overline{12}$ | 12 | 12 | 13 | .. | .. | .. | .. | .. | .. |
| 85 | 2 | 3 | 3 | 4 | 4 | 5 | $\overline{13}$ | $\overline{13}$ | $\overline{12}$ | $\overline{13}$ | $\overline{13}$ | $\overline{12}$ | $\overline{13}$ | $\overline{13}$ | $\overline{12}$ | 12 | 12 | .. | .. | .. | .. | .. | .. | .. |
| 80 | 2 | 3 | 3 | 4 | 4 | $\overline{13}$ | $\overline{13}$ | $\overline{13}$ | $\overline{12}$ | $\overline{12}$ | $\overline{12}$ | $\overline{12}$ | $\overline{12}$ | $\overline{12}$ | $\overline{12}$ | 12 | .. | .. | .. | .. | .. | .. | .. | .. |
| 75 | 2 | 3 | 3 | 4 | 4 | $\overline{13}$ | $\overline{13}$ | $\overline{12}$ | $\overline{12}$ | $\overline{12}$ | $\overline{12}$ | $\overline{12}$ | $\overline{12}$ | $\overline{12}$ | 12 | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| 70 | 2 | 3 | 3 | 4 | 4 | $\overline{13}$ | $\overline{13}$ | $\overline{12}$ | $\overline{12}$ | $\overline{12}$ | $\overline{12}$ | $\overline{11}$ | $\overline{11}$ | 11 | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| 65 | 2 | 3 | 3 | 4 | 4 | $\overline{12}$ | $\overline{12}$ | $\overline{12}$ | $\overline{12}$ | $\overline{12}$ | $\overline{11}$ | $\overline{11}$ | 11 | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| 60 | 11 | 3 | 3 | 4 | 4 | 5 | $\overline{13}$ | $\overline{12}$ | $\overline{11}$ | $\overline{11}$ | $\overline{11}$ | 11 | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| 55 | 11 | 12 | 12 | 12 | 4 | 12 | $\overline{13}$ | $\overline{12}$ | $\overline{12}$ | $\overline{13}$ | 11 | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| 50 | 11 | 12 | 12 | 12 | 12 | 12 | $\overline{13}$ | $\overline{13}$ | $\overline{13}$ | 12 | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| 45 | 2 | 3 | 12 | 12 | 12 | 12 | $\overline{13}$ | $\overline{13}$ | 13 | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| 40 | 2 | 3 | 3 | 3 | 12 | 12 | $\overline{13}$ | 13 | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| 35 | 2 | 3 | 3 | 4 | 4 | 13 | 13 | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| 30 | 2 | 3 | 3 | 4 | 4 | 13 | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| 25 | 2 | 3 | 3 | 4 | 4 | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| 20 | 2 | $\overline{4}$ | 3 | 4 | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| 15 | 2 | 3 | 3 | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| 10 | 2 | 3 | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. |
| 5 | 2 | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. | .. |

General Notes.—The table gives wheel for maximum for any stress which has a triangular influence line.

In case of two unequal segments, the live load approaches on the longer segment except where wheel is overlined, when live load approaches on shorter segment.

When both segments are each greater than 142 ft., advance load on longer segment first, and upon next segment until wheel No. 1 is within 33 feet of the far end of the latter.

## TABLE 4

### Position of Cooper's Loadings for Absolute Maximum Bending Moment in Girder Bridges Without Panels

$S$ = Span in feet.

$c$ = Distance in feet that wheel No. 1 has moved to left beyond centre of span.

$w$ = wheel under which absolute maximum bending moment occurs.

$a$ = distance that $w$ is to left from centre of span.

$b$ = " " $w$ " right " " " "

| $S$ | $c$ | $w$ | $a$ | $b$ |
|---|---|---|---|---|
| 0′ to 8′.5 | 8′.00 | 2 | 0′.00 | .... |
| 8.5 " 11.1 | 9.25 | 2 | 1.25 | .... |
| 11.1 " 18.7 | 13.00 | 3 | 0.00 | .... |
| 18.7 " 27.6 | 14.25 | 3 | 1.25 | .... |
| 27.6 " 34.9 | 13.39 | 3 | 0.39 | .... |
| 34.9 " 38.7 | 17.06 | 4 | .... | 0.94 |
| 38.7 " 48.6 | 18.21 | 4 | 0.21 | .... |
| 48.6 " 53.7 | 19.45 | 4 | 1.45 | .... |
| 53.7 " 58.4 | 74.13 | 13 | 0.13 | .... |
| 58.4 " 63.2 | 75.37 | 13 | 1.37 | .... |
| 63.2 " 70.00 | 74.07 | 13 | 0.07 | .... |

Note.—For spans greater than 70 feet, the maximum centre moment equals the absolute maximum bending moment with an error of less than one per cent.

## TABLE 5

### Position of Cooper's Loadings for Maximum End Shear in Girder Bridges Without Panels

| Span | Direction Load Moves | Position of Load | Location of Maximum Shear |
|---|---|---|---|
| 0′ to 23′ | Right to left | $w_2$ at left end | Left end |
| 23 " 27 | Right to left | $w_5$ at right end | Right end |
| 27 " 46 | Right to left | $w_2$ at left end | Left end |
| 46 " 62 | Right to left | $w_{11}$ at left end | Left end |
| 62 " 400 | Right to left | $w_2$ at left end | Left end |

## TABLE 6

### Position of Cooper's Loadings for Maximum Shear in Panels of Girder and Truss Bridges

| Number of Panels | Panel | Panel Length in Feet | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 22 | 23 | 24 | 25 | 26 | 27 | 28 | 29 | 30 | 31 | 32 | 33 | 34 | 35 |
| 6. . . . . . . . . . . | 0–1 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 5 | 5 | 5 | 5 |
| | 1–2 | 3 | 3 | 3 | 3 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 |
| | 2–3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 4 |
| | 3–4 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 3 | 3 | 3 | 3 | 3 |
| | 4–5 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 |
| 7. . . . . . . . . . . | 0–1 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 5 | 5 | 5 |
| | 1–2 | 3 | 3 | 3 | 3 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 |
| | 2–3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 4 | 4 |
| | 3–4 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 |
| | 4–5 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 3 | 3 |
| | 5–6 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 |
| 8. . . . . . . . . . . | 0–1 | 3 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 5 | 5 | 5 |
| | 1–2 | 3 | 3 | 3 | 3 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 |
| | 2–3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 4 | 4 | 4 | 4 | 4 |
| | 3–4 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 |
| | 4–5 | 2 | 2 | 2 | 2 | 2 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 |
| | 5–6 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 |
| | 6–7 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 |
| 9. . . . . . . . . . . | 0–1 | 3 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 5 | 5 |
| | 1–2 | 3 | 3 | 3 | 3 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 |
| | 2–3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 4 | 4 | 4 | 4 | 4 | 4 |
| | 3–4 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 |
| | 4–5 | 2 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 |
| | 5–6 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 3 | 3 | 3 | 3 | 3 |
| | 6–7 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 |
| | 7–8 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 |
| 10. . . . . . . . . . . | 0–1 | 3 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 5 | 5 |
| | 1–2 | 3 | 3 | 3 | 3 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 |
| | 2–3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 4 | 4 | 4 | 4 | 4 | 4 |
| | 3–4 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 4 |
| | 4–5 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 |
| | 5–6 | 2 | 2 | 2 | 2 | 2 | 2 | 3 | 3 | 3 | 3 | 3 | 3 | 3 | 3 |
| | 6–7 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 |
| | 7–8 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 | 2 |
| | 8–9 | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 1 | 2 | 2 | 2 | 2 | 2 | 2 |

**Note.—Place tabulated wheel at right end of corresponding panel with locomotive advancing toward left.**

TABLE 7

MAXIMUM MOMENTS, SHEARS, AND PIER REACTIONS FOR COOPER'S STANDARD LOADINGS

(Figures for One Rail)

| Span | *E*40 | | | | | *E*50 | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Max. Moment | Max. Shears | | | Max. Pier React. | Max. Moment | Max. Shears | | | Max. Pier React. |
| | | End | ¼ Pt. | Cent. | | | End | ¼ Pt. | Cent. | |
| 10...... | 56.3 | 30.0 | 20.0 | 10.0 | 40.0 | 70.4 | 37.5 | 25.0 | 12.5 | 50.0 |
| 11...... | 65.7 | 32.7 | 20.9 | 10.9 | 43.7 | 82.1 | 40.9 | 26.1 | 13.6 | 54.5 |
| 12...... | 80.0 | 35.0 | 21.7 | 11.7 | 46.7 | 100.0 | 43.8 | 27.1 | 14.6 | 58.4 |
| 13...... | 95.0 | 36.9 | 22.3 | 12.3 | 49.2 | 118.8 | 46.2 | 27.9 | 15.4 | 61.6 |
| 14...... | 110.0 | 38.6 | 23.6 | 12.9 | 52.2 | 137.5 | 48.2 | 29.5 | 16.2 | 65.2 |
| 15...... | 125.0 | 40.0 | 25.0 | 13.3 | 54.7 | 156.3 | 50.0 | 31.3 | 16.6 | 68.3 |
| 16...... | 140.0 | 42.5 | 26.3 | 13.7 | 56.9 | 175.0 | 53.1 | 32.9 | 17.1 | 71.1 |
| 17...... | 155.0 | 44.7 | 27.4 | 13.8 | 58.8 | 193.8 | 55.9 | 34.3 | 17.3 | 73.5 |
| 18...... | 170.0 | 46.7 | 28.3 | 13.9 | 60.7 | 212.5 | 58.3 | 35.4 | 17.4 | 75.9 |
| 19...... | 186.6 | 48.4 | 29.2 | 14.0 | 62.9 | 233.3 | 60.5 | 36.5 | 17.5 | 78.6 |
| 20...... | 206.3 | 50.0 | 30.0 | 14.0 | 65.6 | 257.9 | 62.5 | 37.5 | 17.5 | 81.9 |
| 21...... | 226.0 | 51.4 | 31.4 | 14.5 | 68.0 | 282.5 | 64.3 | 39.2 | 18.1 | 84.9 |
| 22...... | 245.7 | 52.7 | 32.7 | 15.0 | 70.2 | 307.1 | 65.9 | 40.9 | 18.8 | 87.6 |
| 23...... | 265.4 | 53.9 | 33.9 | 15.4 | 72.2 | 331.8 | 67.4 | 42.4 | 19.3 | 90.2 |
| 24...... | 285.2 | 55.4 | 35.0 | 15.8 | 74.0 | 356.5 | 69.3 | 43.8 | 19.8 | 92.4 |
| 25...... | 305.0 | 56.8 | 36.0 | 16.2 | 75.7 | 381.3 | 71.0 | 45.0 | 20.2 | 94.6 |
| 26...... | 324.8 | 58.1 | 36.9 | 16.5 | 77.7 | 406.0 | 72.6 | 46.1 | 20.6 | 97.1 |
| 27...... | 344.6 | 59.2 | 37.8 | 16.9 | 80.2 | 430.8 | 74.0 | 47.2 | 21.1 | 100.1 |
| 28...... | 365.5 | 60.4 | 38.6 | 17.1 | 82.3 | 456.9 | 75.5 | 48.2 | 21.4 | 102.8 |
| 29...... | 388.0 | 61.6 | 39.3 | 17.4 | 84.4 | 485.0 | 76.9 | 49.1 | 21.8 | 105.4 |
| 30...... | 410.5 | 63.0 | 40.0 | 17.7 | 86.3 | 513.0 | 78.8 | 50.0 | 22.1 | 107.9 |
| 31...... | 432.9 | 64.4 | 40.7 | 18.2 | 88.5 | 541.1 | 80.5 | 50.9 | 22.7 | 110.6 |
| 32...... | 455.4 | 65.7 | 41.3 | 18.8 | 91.0 | 569.3 | 82.1 | 51.8 | 23.4 | 113.7 |
| 33...... | 477.9 | 66.9 | 42.0 | 19.2 | 93.3 | 597.4 | 83.7 | 52.5 | 24.0 | 116.7 |
| 34...... | 500.6 | 68.1 | 42.8 | 19.7 | 95.5 | 625.8 | 85.1 | 53.5 | 24.6 | 119.4 |
| 35...... | 523.0 | 69.2 | 43.5 | 20.1 | 97.5 | 653.8 | 86.5 | 54.4 | 25.1 | 122.0 |
| 36...... | 548.6 | 70.6 | 44.1 | 20.6 | 99.6 | 685.8 | 88.2 | 55.1 | 25.8 | 124.4 |
| 37...... | 574.3 | 71.9 | 44.8 | 21.0 | 101.5 | 717.9 | 89.8 | 56.0 | 26.2 | 126.9 |
| 38...... | 600.0 | 73.1 | 45.4 | 21.3 | 103.7 | 750.0 | 91.4 | 56.7 | 26.6 | 129.7 |
| 39...... | 626.6 | 74.3 | 46.0 | 21.7 | 105.9 | 783.3 | 92.9 | 57.5 | 27.1 | 132.3 |
| 40...... | 655.6 | 75.4 | 46.8 | 22.0 | 108.0 | 819.5 | 94.3 | 58.5 | 27.5 | 135.0 |
| 41...... | 684.6 | 76.8 | 47.5 | 22.3 | 110.0 | 855.8 | 96.0 | 59.4 | 27.9 | 137.6 |
| 42...... | 713.6 | 78.4 | 48.2 | 22.6 | 112.1 | 892.0 | 97.6 | 60.2 | 28.3 | 140.2 |
| 43...... | 742.6 | 79.4 | 48.9 | 22.9 | 114.3 | 928.3 | 99.2 | 61.1 | 28.6 | 142.9 |
| 44...... | 771.6 | 80.6 | 49.5 | 23.2 | 116.5 | 964.5 | 100.7 | 61.9 | 29.0 | 145.6 |
| 45...... | 800.6 | 81.7 | 50.1 | 23.4 | 118.6 | 1000.8 | 102.1 | 62.6 | 29.3 | 148.3 |
| 46...... | 829.8 | 82.8 | 50.7 | 23.7 | 120.7 | 1037.3 | 103.5 | 63.4 | 29.6 | 150.9 |
| 47...... | 858.6 | 83.8 | 51.4 | 23.9 | 122.7 | 1073.3 | 104.9 | 64.2 | 29.9 | 153.4 |
| 48...... | 887.6 | 85.0 | 52.1 | 24.2 | 124.8 | 1109.5 | 106.3 | 65.1 | 30.2 | 156.0 |
| 49...... | 918.8 | 86.1 | 52.8 | 24.5 | 126.8 | 1148.5 | 107.7 | 66.0 | 30.6 | 158.5 |
| 50...... | 950.9 | 87.2 | 53.5 | 24.9 | 128.7 | 1188.6 | 109.0 | 66.8 | 31.1 | 161.0 |
| 51...... | 983.1 | 88.4 | 54.1 | 25.2 | 131.0 | 1228.9 | 110.4 | 67.6 | 31.5 | 163.6 |
| 52...... | 1015.2 | 89.3 | 54.8 | 25.5 | 133.3 | 1269.0 | 111.8 | 68.5 | 31.9 | 166.6 |
| 53...... | 1047.4 | 90.5 | 55.4 | 25.8 | 135.6 | 1309.2 | 113.1 | 69.2 | 32.3 | 169.6 |

TABLE 7.—*Continued*

MAXIMUM MOMENTS, SHEARS, AND PIER REACTIONS FOR COOPER'S STANDARD LOADINGS

(Figures for One Rail)

| Span | *E*40 | | | | | *E*50 | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Max. Moment | Max. Shears End | Max. Shears ¼ Pt. | Max. Shears Cent. | Max. Pier React. | Max. Moment | Max. Shears End | Max. Shears ¼ Pt. | Max. Shears Cent. | Max. Pier React. |
| 54...... | 1081.4 | 91.5 | 56.1 | 26.1 | 138.0 | 1351.8 | 114.5 | 70.1 | 32.6 | 172.5 |
| 55...... | 1116.9 | 92.6 | 56.8 | 26.4 | 140.3 | 1396.1 | 115.8 | 71.0 | 33.0 | 175.4 |
| 56...... | 1152.4 | 93.7 | 57.5 | 26.6 | 142.7 | 1440.5 | 117.2 | 71.8 | 33.3 | 178.5 |
| 57...... | 1187.9 | 94.8 | 58.2 | 26.9 | 145.4 | 1484.9 | 118.5 | 72.7 | 33.6 | 181.8 |
| 58...... | 1223.4 | 95.9 | 58.8 | 27.2 | 148.1 | 1529.2 | 119.8 | 73.5 | 34.0 | 185.1 |
| 59...... | 1261.0 | 97.0 | 59.5 | 27.5 | 150.6 | 1576.2 | 121.2 | 74.4 | 34.4 | 188.4 |
| 60...... | 1299.6 | 98.0 | 60.1 | 27.9 | 153.2 | 1624.5 | 122.5 | 75.2 | 34.9 | 191.5 |
| 61...... | 1338.3 | 99.2 | 60.7 | 28.2 | 155.7 | 1672.9 | 123.9 | 76.0 | 35.2 | 194.7 |
| 62...... | 1377.0 | 100.1 | 61.3 | 28.5 | 158.2 | 1721.2 | 125.2 | 76.6 | 35.6 | 197.7 |
| 63...... | 1415.6 | 101.3 | 61.8 | 28.8 | 160.4 | 1769.5 | 126.6 | 77.4 | 36.0 | 200.7 |
| 64...... | 1455.5 | 102.6 | 62.4 | 29.1 | 162.6 | 1819.4 | 128.2 | 78.0 | 36.4 | 203.6 |
| 65...... | 1497.5 | 103.8 | 63.0 | 29.4 | 165.2 | 1871.9 | 129.7 | 78.8 | 36.8 | 206.7 |
| 66...... | 1539.5 | 105.0 | 63.6 | 29.7 | 167.8 | 1924.4 | 131.2 | 79.5 | 37.1 | 209.7 |
| 67...... | 1581.5 | 106.4 | 64.2 | 30.0 | 170.1 | 1976.9 | 133.0 | 80.3 | 37.5 | 212.7 |
| 68...... | 1623.5 | 107.8 | 64.8 | 30.2 | 172.5 | 2029.4 | 134.8 | 81.0 | 37.8 | 215.6 |
| 69...... | 1665.5 | 109.2 | 65.4 | 30.5 | 174.8 | 2081.9 | 136.5 | 81.7 | 38.1 | 218.5 |
| 70...... | 1707.5 | 110.5 | 65.9 | 30.7 | 177.1 | 2134.4 | 138.1 | 82.4 | 38.4 | 221.3 |
| 71...... | 1749.3 | 111.8 | 66.5 | 31.1 | 179.3 | 2186.6 | 139.8 | 83.1 | 38.8 | 224.1 |
| 72...... | 1793.0 | 113.3 | 67.0 | 31.4 | 181.5 | 2241.2 | 141.7 | 83.8 | 39.2 | 226.9 |
| 73...... | 1833.9 | 114.8 | 67.5 | 31.7 | 183.7 | 2292.4 | 143.5 | 84.4 | 39.6 | 229.6 |
| 74...... | 1879.2 | 116.3 | 68.0 | 32.0 | 186.0 | 2349.0 | 145.3 | 85.0 | 40.0 | 232.4 |
| 75...... | 1925.8 | 117.7 | 68.6 | 32.3 | 188.2 | 2407.3 | 147.1 | 85.7 | 40.4 | 235.2 |
| 76...... | 1972.0 | 119.1 | 69.2 | 32.6 | 190.4 | 2465.0 | 148.8 | 86.5 | 40.8 | 238.0 |
| 77...... | 2019.1 | 120.4 | 69.9 | 32.9 | 192.5 | 2523.9 | 150.5 | 87.4 | 41.1 | 240.7 |
| 78...... | 2065.0 | 121.7 | 70.5 | 33.2 | 194.7 | 2581.2 | 152.1 | 88.2 | 41.5 | 243.3 |
| 79...... | 2112.3 | 123.0 | 71.1 | 33.4 | 196.8 | 2640.4 | 153.8 | 88.9 | 41.7 | 245.9 |
| 80...... | 2160.5 | 124.2 | 71.7 | 33.7 | 198.9 | 2700.6 | 155.3 | 89.6 | 42.1 | 248.6 |
| 81...... | 2207.7 | 125.6 | 72.3 | 34.0 | 200.9 | 2759.6 | 157.0 | 90.4 | 42.5 | 251.1 |
| 82...... | 2256.7 | 126.9 | 73.0 | 34.4 | 203.0 | 2820.9 | 158.6 | 91.2 | 43.0 | 253.6 |
| 83...... | 2306.5 | 128.2 | 73.7 | 34.7 | 205.0 | 2883.1 | 160.3 | 92.1 | 43.4 | 256.1 |
| 84...... | 2356.3 | 129.5 | 74.4 | 35.0 | 206.9 | 2945.4 | 161.8 | 93.0 | 43.7 | 258.7 |
| 85...... | 2406.9 | 130.7 | 75.1 | 35.3 | 208.9 | 3008.6 | 163.4 | 93.9 | 44.1 | 260.8 |
| 86...... | 2459.6 | 132.1 | 75.8 | 35.6 | 210.8 | 3074.5 | 165.1 | 94.3 | 44.5 | 263.0 |
| 87...... | 2510.6 | 133.4 | 76.5 | 35.9 | 212.8 | 3138.3 | 166.8 | 95.7 | 44.9 | 265.6 |
| 88...... | 2564.2 | 134.7 | 77.1 | 36.2 | 214.7 | 3205.3 | 168.4 | 96.5 | 45.2 | 268.3 |
| 89...... | 2615.9 | 136.0 | 77.9 | 36.5 | 216.7 | 3269.9 | 170.0 | 97.4 | 45.6 | 270.8 |
| 90...... | 2670.5 | 137.2 | 78.7 | 36.7 | 218.6 | 3338.1 | 171.5 | 98.4 | 45.9 | 273.2 |
| 91...... | 2723.0 | 138.5 | 79.5 | 37.0 | 220.6 | 3403.7 | 173.1 | 99.4 | 46.2 | 275.6 |
| 92...... | 2776.7 | 139.8 | 80.3 | 37.3 | 222.5 | 3470.9 | 174.7 | 100.4 | 46.6 | 278.0 |
| 93...... | 2831.5 | 141.1 | 81.0 | 37.5 | 224.4 | 3539.3 | 176.4 | 101.2 | 46.9 | 280.3 |
| 94...... | 2885.3 | 142.4 | 81.7 | 37.8 | 226.3 | 3606.6 | 178.0 | 102.1 | 47.3 | 282.7 |
| 95...... | 2939.5 | 143.6 | 82.5 | 38.0 | 228.1 | 3674.3 | 179.5 | 103.1 | 47.5 | 285.1 |
| 96...... | 2994.5 | 144.8 | 83.3 | 38.3 | 230.0 | 3743.1 | 181.0 | 104.1 | 47.9 | 287.5 |
| 97...... | 3049.0 | 146.2 | 84.2 | 38.5 | 231.8 | 3811.2 | 182.7 | 105.1 | 48.1 | 289.7 |

TABLE 7.—*Continued*

MAXIMUM MOMENTS, SHEARS AND PIER REACTIONS FOR COOPER'S STANDARD LOADINGS

(Figures for One Rail)

| Span | *E*40 | | | | | *E*50 | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Max. Moment | Max. Shears | | | Max. Pier React. | Max. Moment | Max. Shears | | | Max. Pier React. |
| | | End | ¼ Pt. | Cent. | | | End | ¼ Pt. | Cent. | |
| 98...... | 3106.5 | 147.5 | 85.0 | 38.8 | 233.6 | 3883.1 | 184.3 | 106.2 | 48.5 | 292.0 |
| 99...... | 3162.3 | 148.8 | 85.8 | 39.1 | 235.4 | 3952.9 | 186.0 | 107.2 | 48.9 | 294.2 |
| 100...... | 3219.9 | 150.0 | 86.6 | 39.4 | 237.2 | 4024.9 | 187.5 | 108.2 | 49.2 | 296.5 |
| 101...... | 3277.6 | 151.2 | 87.3 | 39.6 | 238.9 | 4097.0 | 189.0 | 109.1 | 49.5 | 298.6 |
| 102...... | 3335.9 | 152.4 | 88.1 | 39.9 | 240.6 | 4169.9 | 190.6 | 110.1 | 49.9 | 300.8 |
| 103...... | 3410.6 | 153.7 | 88.8 | 40.1 | 242.4 | 4263.3 | 192.1 | 111.0 | 50.1 | 303.0 |
| 104...... | 3475.2 | 154.9 | 89.5 | 40.4 | 244.2 | 4344.0 | 193.6 | 111.9 | 50.5 | 305.3 |
| 105...... | 3537.6 | 156.1 | 90.3 | 40.6 | 246.0 | 4422.0 | 195.1 | 112.7 | 50.7 | 307.5 |
| 106...... | 3600.3 | 157.3 | 90.9 | 40.9 | 247.8 | 4500.4 | 196.6 | 113.6 | 51.1 | 309.8 |
| 107...... | 3666.6 | 158.5 | 91.7 | 41.1 | 249.6 | 4583.3 | 198.1 | 114.5 | 51.5 | 312.0 |
| 108...... | 3745.3 | 159.6 | 92.4 | 41.3 | 251.4 | 4681.6 | 199.5 | 115.5 | 51.7 | 314.2 |
| 109...... | 3818.4 | 160.8 | 93.2 | 41.6 | 253.1 | 4773.0 | 201.0 | 116.4 | 52.0 | 316.3 |
| 110...... | 3886.8 | 162.0 | 93.9 | 41.8 | 254.8 | 4858.5 | 202.5 | 117.4 | 52.3 | 318.5 |
| 111...... | 3958.2 | 163.2 | 94.6 | 42.0 | 256.5 | 4947.7 | 204.0 | 118.2 | 52.5 | 320.7 |
| 112...... | 4026.9 | 164.4 | 95.3 | 42.2 | 258.2 | 5033.6 | 205.5 | 119.1 | 52.7 | 322.8 |
| 113...... | 4099.0 | 165.5 | 96.0 | 42.5 | 259.9 | 5123.8 | 207.0 | 120.0 | 53.1 | 324.9 |
| 114...... | 4172.0 | 166.7 | 96.8 | 42.8 | 261.6 | 5215.0 | 208.4 | 121.0 | 53.5 | 327.0 |
| 115...... | 4245.0 | 167.9 | 97.5 | 43.1 | 263.3 | 5306.2 | 209.9 | 121.9 | 53.9 | 329.0 |
| 116...... | 4318.8 | 169.0 | 98.3 | 43.4 | 264.9 | 5398.5 | 211.3 | 122.9 | 54.2 | 331.1 |
| 117...... | 4389.5 | 170.2 | 99.0 | 43.7 | 266.7 | 5486.9 | 212.8 | 123.7 | 54.6 | 333.3 |
| 118...... | 4463.8 | 171.4 | 99.7 | 43.9 | 268.5 | 5579.7 | 214.2 | 124.6 | 54.9 | 335.6 |
| 119...... | 4538.8 | 172.5 | 100.4 | 44.2 | 270.2 | 5673.5 | 215.7 | 125.5 | 55.3 | 337.8 |
| 120...... | 4614.1 | 173.7 | 101.1 | 44.5 | 272.0 | 5767.6 | 217.1 | 126.4 | 55.6 | 340.0 |
| 121...... | 4686.5 | 174.8 | 101.8 | 44.7 | 273.8 | 5858.1 | 218.6 | 127.2 | 55.9 | 342.2 |
| 122...... | 4762.7 | 176.0 | 102.5 | 45.0 | 275.6 | 5953.4 | 220.0 | 128.1 | 56.2 | 344.5 |
| 123...... | 4836.2 | 177.1 | 103.2 | 45.3 | 277.4 | 6045.2 | 221.4 | 129.0 | 56.5 | 346.7 |
| 124...... | 4917.4 | 178.3 | 104.0 | 45.7 | 279.2 | 6146.7 | 222.8 | 130.0 | 57.0 | 349.0 |
| 125...... | 4996.4 | 179.4 | 104.7 | 46.0 | 281.0 | 6245.5 | 224.2 | 130.9 | 57.5 | 351.2 |
| 150...... | 7062.3 | 207.4 | 121.8 | 54.4 | 325.4 | 8827.9 | 259.2 | 152.2 | 68.0 | 406.7 |
| 175...... | 9352.5 | 234.5 | 138.3 | 62.5 | 371.7 | 11690.6 | 293.1 | 172.9 | 78.2 | 464.6 |
| 200...... | 11873.0 | 261.0 | 153.4 | 70.4 | 419.0 | 14841.2 | 326.3 | 191.8 | 88.0 | 523.8 |
| 250...... | 17592.5 | 313.2 | 183.7 | 85.0 | 515.2 | 21990.6 | 391.5 | 229.6 | 106.3 | 644.0 |

NOTES.—Moments are given in thousand foot-pounds.

Shears are given in thousand pounds.

Pier reactions are given in thousand pounds and are for piers between two spans each equal to the tabulated span.

## TABLE 8

### Maximum Moments for Truss Bridges—Cooper's $E50$ for One Rail

Moments Given in Thousands of Foot-Pounds

Panel Points 0 1 2 3 4 5 6 7 8 9

| Panels in Truss | Panel Points | Panel Lengths 8′ 0″ | 8′ 6″ | 9′ 0″ | 9′ 6″ | 10′ 0″ | 10′ 6″ | 11′ 0″ | 11′ 6″ | 12′ 0″ | 12′ 6″ | 13′ 0″ | 13′ 6″ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 3 | 1 | 325 | 359 | 392 | 425 | 464 | 503 | 541 | 580 | 619 | 661 | 707 | 755 |
| 4 | 1 | 433 | 483 | 533 | 582 | 632 | 688 | 743 | 799 | 859 | 918 | 982 | 1046 |
| | 2 | 569 | 625 | 683 | 747 | 819 | 892 | 964 | 1037 | 1110 | 1189 | 1269 | 1352 |
| 5 | 1 | 540 | 599 | 662 | 728 | 794 | 861 | 930 | 1001 | 1071 | 1140 | 1217 | 1298 |
| | 2 | 790 | 877 | 964 | 1051 | 1149 | 1255 | 1361 | 1468 | 1574 | 1675 | 1792 | 1910 |
| 6 | 1 | 641 | 710 | 784 | 859 | 937 | 1017 | 1100 | 1186 | 1280 | 1375 | 1485 | 1600 |
| | 2 | 1008 | 1115 | 1228 | 1347 | 1466 | 1587 | 1719 | 1857 | 1997 | 2135 | 2289 | 2451 |
| | 3 | 1109 | 1221 | 1351 | 1484 | 1618 | 1767 | 1925 | 2070 | 2240 | 2407 | 2581 | 2760 |
| 7 | 1 | 731 | 812 | 896 | 984 | 1080 | 1184 | 1293 | 1411 | 1530 | 1645 | 1775 | 1906 |
| | 2 | 1215 | 1344 | 1477 | 1615 | 1758 | 1904 | 2070 | 2252 | 2441 | 2642 | 2849 | 3050 |
| | 3 | 1425 | 1577 | 1739 | 1910 | 2086 | 2269 | 2465 | 2667 | 2879 | 3100 | 3332 | 3560 |
| 8 | 1 | 819 | 915 | 1021 | 1133 | 1254 | 1375 | 1501 | 1631 | 1776 | 1900 | 2047 | 2200 |
| | 2 | 1402 | 1553 | 1709 | 1872 | 2061 | 2273 | 2490 | 2708 | 2933 | 3165 | 3405 | 3649 |
| | 3 | 1716 | 1899 | 2100 | 2311 | 2529 | 2752 | 2991 | 3241 | 3498 | 3775 | 4078 | 4383 |
| | 4 | 1819 | 2030 | 2240 | 2465 | 2700 | 2946 | 3205 | 3471 | 3743 | 4025 | 4344 | 4681 |
| 9 | 1 | 621 | 1039 | 1162 | 1287 | 1418 | 1556 | 1697 | 1844 | 1997 | 2145 | 2309 | 2475 |
| | 2 | 1583 | 1764 | 1960 | 2179 | 2405 | 2642 | 2888 | 3139 | 3400 | 3670 | 3946 | 4224 |
| | 3 | 1997 | 2215 | 2451 | 2700 | 2986 | 3276 | 3570 | 3877 | 4194 | 4532 | 4887 | 5242 |
| | 4 | 2208 | 2459 | 2719 | 2997 | 3291 | 3592 | 3899 | 4226 | 4588 | 4970 | 5370 | 5770 |

| Panels in Truss | Panel Points | Panel Lengths 14′ 0″ | 14′ 6″ | 15′ 0″ | 15′ 6″ | 16′ 0″ | 16′ 6″ | 17′ 0″ | 17′ 6″ | 18′ 0″ | 18′ 6″ | 19′ 0″ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 3 | 1 | 803 | 850 | 900 | 952 | 1008 | 1060 | 1115 | 1170 | 1228 | 1285 | 1347 |
| 4 | 1 | 1115 | 1183 | 1255 | 1325 | 1402 | 1463 | 1553 | 1614 | 1709 | 1776 | 1872 |
| | 2 | 1441 | 1529 | 1624 | 1721 | 1820 | 1924 | 2030 | 2134 | 2240 | 2349 | 2465 |
| 5 | 1 | 1389 | 1480 | 1581 | 1680 | 1788 | 1896 | 2010 | 2123 | 2242 | 2355 | 2477 |
| | 2 | 2047 | 2177 | 2310 | 2440 | 2581 | 2725 | 2881 | 3030 | 3190 | 3350 | 3518 |
| 6 | 1 | 1724 | 1840 | 1965 | 2090 | 2221 | 2352 | 2489 | 2626 | 2769 | 2910 | 3062 |
| | 2 | 2616 | 2792 | 2986 | 3175 | 3372 | 3570 | 3775 | 3978 | 4194 | 4415 | 4650 |
| | 3 | 2946 | 3138 | 3338 | 3539 | 3742 | 3953 | 4170 | 4422 | 4681 | 4948 | 5215 |
| 7 | 1 | 2047 | 2185 | 2332 | 2480 | 2634 | 2787 | 2945 | 3104 | 3268 | 3434 | 3605 |
| | 2 | 3263 | 3485 | 3723 | 3958 | 4202 | 4450 | 4705 | 4958 | 5218 | 5480 | 5748 |
| | 3 | 3802 | 4040 | 4310 | 4595 | 4898 | 5200 | 5509 | 5815 | 6135 | 6460 | 6800 |
| 8 | 1 | 2358 | 2516 | 2681 | 2846 | 3019 | 3190 | 3372 | 3553 | 3741 | 3930 | 4125 |
| | 2 | 3900 | 4165 | 4436 | 4710 | 4994 | 5280 | 5576 | 5873 | 6180 | 6487 | 6805 |
| | 3 | 4710 | 5040 | 5380 | 5720 | 6072 | 6430 | 6806 | 7180 | 7573 | 7985 | 8369 |
| | 4 | 5034 | 5398 | 5768 | 6147 | 6516 | 6915 | 7331 | 7740 | 8163 | 8595 | 9043 |
| 9 | 1 | 2651 | 2828 | 3012 | 3196 | 3389 | 3583 | 3785 | 3987 | 4198 | 4410 | 4629 |
| | 2 | 4512 | 4804 | 5107 | 5420 | 5747 | 6074 | 6414 | 6755 | 7108 | 7463 | 7830 |
| | 3 | 5617 | 5993 | 6390 | 6790 | 7204 | 7620 | 8054 | 8496 | 8959 | 9415 | 9892 |
| | 4 | 6187 | 6610 | 7040 | 7485 | 7966 | 6460 | 8980 | 9490 | 10010 | 10530 | 11065 |

## TABLE 8.—*Continued*

### Maximum Moments for Truss Bridges—Cooper's *E*50 for One Rail

### Moments Given in Thousands of Foot-Pounds

Panel Points 0 1 2 3 4 5 6 7 8 9

| Panels in Truss | Panel Points | Panel Lengths | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 19′ 6″ | 20′ 0″ | 20′ 6″ | 21′ 0″ | 21′ 6″ | 22′ 0″ | 22′ 6″ | 23′ 0″ | 23′ 6″ | 24′ 0″ | 24′ 6″ |
| 3 | 1 | 1404 | 1466 | 1527 | 1587 | 1653 | 1719 | 1788 | 1857 | 1927 | 1997 | 2066 |
| 4 | 1 | 1958 | 2061 | 2166 | 2273 | 2380 | 2490 | 2597 | 2708 | 2819 | 2933 | 3046 |
| | 2 | 2581 | 2700 | 2821 | 2946 | 3074 | 3205 | 3338 | 3471 | 3607 | 3743 | 3883 |
| 5 | 1 | 2600 | 2731 | 2864 | 3001 | 3138 | 3279 | 3418 | 3562 | 3705 | 3852 | 3999 |
| | 2 | 3685 | 3943 | 4144 | 4347 | 4555 | 4767 | 4978 | 5193 | 5415 | 5640 | 5865 |
| 6 | 1 | 3210 | 3362 | 3516 | 3678 | 3840 | 4008 | 4175 | 4349 | 4522 | 4700 | 4878 |
| | 2 | 4885 | 5256 | 5501 | 5750 | 5998 | 6250 | 6501 | 6756 | 7011 | 7270 | 7525 |
| | 3 | 5487 | 5746 | 6028 | 6321 | 6617 | 6921 | 7228 | 7538 | 7850 | 8166 | 8491 |
| 7 | 1 | 3778 | 3955 | 4130 | 4317 | 4505 | 4702 | 4897 | 5100 | 5303 | 5512 | 5721 |
| | 2 | 6025 | 6326 | 6613 | 6914 | 7215 | 7530 | 7845 | 8173 | 8503 | 8842 | 9182 |
| | 3 | 7140 | 7646 | 7990 | 8347 | 8710 | 9079 | 9448 | 9826 | 10207 | 10609 | 11017 |
| 8 | 1 | 4320 | 4525 | 4727 | 4939 | 5150 | 5373 | 5592 | 5829 | 6061 | 6300 | 6540 |
| | 2 | 7125 | 7458 | 7805 | 8162 | 8520 | 8890 | 9260 | 9640 | 10030 | 10430 | 10832 |
| | 3 | 8780 | 9234 | 9630 | 10070 | 10515 | 10993 | 11475 | 11976 | 12472 | 12981 | 13490 |
| | 4 | 94[illegible]0 | 9943 | 10396 | 10862 | 11317 | 11805 | 12288 | 12790 | 13287 | 13795 | 14300 |
| 9 | 1 | 4850 | 5079 | 5308 | 5545 | 5780 | 6030 | 6280 | 6542 | 6804 | 7074 | 7344 |
| | 2 | 8198 | 8578 | 8970 | 9378 | 9790 | 10216 | 10640 | 11082 | 11525 | 11985 | 12448 |
| | 3 | 10372 | 10880 | 11375 | 11900 | 12425 | 12978 | 13535 | 14118 | 14705 | 15308 | 15910 |
| | 4 | 11605 | 12172 | 12735 | 13310 | 13880 | 14472 | 15068 | 15684 | 16300 | 16930 | 17560 |

| Panels in Truss | Panel Points | Panel Lengths | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 25′ 0″ | 25′ 6″ | 26′ 0″ | 26′ 6″ | 27′ 0″ | 27′ 6″ | 28′ 0″ | 28′ 6″ | 29′ 0″ | 29′ 6″ | 30′ 0″ |
| 3 | 1 | 2135 | 2215 | 2289 | 2370 | 2451 | 2534 | 2616 | 2700 | 2792 | 2889 | 2986 |
| 4 | 1 | 3165 | 3282 | 3405 | 3526 | 3649 | 3774 | 3900 | 4031 | 4165 | 4300 | 4436 |
| | 2 | 4025 | 4170 | 4344 | 4501 | 4681 | 4858 | 5034 | 5215 | 5398 | 5580 | 5768 |
| 5 | 1 | 4150 | 4301 | 4456 | 4611 | 4770 | 4929 | 5092 | 5255 | 5422 | 5589 | 5760 |
| | 2 | 6093 | 6371 | 6552 | 6783 | 7017 | 7250 | 7492 | 7736 | 7984 | 8232 | 8482 |
| 6 | 1 | 5061 | 5245 | 5433 | 5622 | 5816 | 6010 | 6208 | 6408 | 6612 | 6817 | 7026 |
| | 2 | 7794 | 8068 | 8352 | 8654 | 8960 | 9268 | 9580 | 9897 | 10218 | 10547 | 10880 |
| | 3 | 8821 | 9153 | 9490 | 9828 | 10170 | 10514 | 10862 | 11208 | 11565 | 11925 | 12296 |
| 7 | 1 | 5936 | 6151 | 6373 | 6595 | 6823 | 7051 | 7286 | 7521 | 7762 | 8003 | 8250 |
| | 2 | 9530 | 9875 | 10236 | 10600 | 10980 | 11357 | 11742 | 12125 | 12520 | 12918 | 13330 |
| | 3 | 11444 | 11870 | 12312 | 12752 | 13203 | 13653 | 14112 | 14571 | 15039 | 15507 | 15984 |
| 8 | 1 | 6787 | 7035 | 7289 | 7540 | 7806 | 8069 | 8338 | 8608 | 8887 | 9165 | 9450 |
| | 2 | 11244 | 11655 | 12080 | 12508 | 12950 | 13392 | 13850 | 14308 | 14780 | 15250 | 15730 |
| | 3 | 14010 | 14528 | 15063 | 15605 | 16163 | 16718 | 17285 | 17852 | 18431 | 19010 | 19600 |
| | 4 | 14820 | 15340 | 15875 | 16413 | 16965 | 17514 | 18075 | 18635 | 19210 | 19795 | 20406 |
| 9 | 1 | 7622 | 7900 | 8188 | 8477 | 8774 | 9070 | 9376 | 9686 | 9996 | 10310 | 10633 |
| | 2 | 12925 | 13400 | 13890 | 14380 | 14888 | 15400 | 15930 | 16460 | 17005 | 17547 | 18100 |
| | 3 | 16528 | 17145 | 17778 | 18414 | 19070 | 19730 | 20405 | 21080 | 21770 | 22461 | 23168 |
| | 4 | 18205 | 18850 | 19515 | 20180 | 20870 | 21557 | 22260 | 22955 | 23678 | 24405 | 25170 |

## TABLE 8.—*Continued*

### Maximum Moments for Truss Bridges—Cooper's *E*50 for One Rail

Moments Given in Thousands of Foot-Pounds

Panel Points 0 1 2 3 4 5 6 7 8 9

| Panels in Truss | Panel Points | Panel Lengths | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 30′ 6″ | 31′ 0″ | 31′ 6″ | 32′ 0″ | 32′ 6″ | 33′ 0″ | 33′ 6″ | 34′ 0″ | 34′ 6″ | 35′ 0″ | 35′ 6″ |
| 3 | 1 | 3080 | 3175 | 3276 | 3372 | 3471 | 3570 | 3672 | 3775 | 3877 | 3978 | 4080 |
| 4 | 1 | 4573 | 4710 | 4852 | 4994 | 5137 | 5280 | 5428 | 5576 | 5725 | 5873 | 5923 |
| | 2 | 5957 | 6147 | 6332 | 6516 | 6715 | 6915 | 7123 | 7331 | 7535 | 7740 | 7950 |
| 5 | 1 | 5937 | 6113 | 6295 | 6477 | 6678 | 6849 | 7039 | 7228 | 7423 | 7617 | 7814 |
| | 2 | 8734 | 8986 | 9241 | 9496 | 9749 | 10012 | 10291 | 10590 | 10891 | 11192 | 11495 |
| 6 | 1 | 7238 | 7450 | 7671 | 7892 | 8120 | 8347 | 8581 | 8812 | 9050 | 9288 | 9628 |
| | 2 | 11219 | 11558 | 11903 | 12248 | 12684 | 12979 | 13354 | 13729 | 14120 | 14510 | 14902 |
| | 3 | 12668 | 13040 | 13418 | 13796 | 14180 | 14563 | 14952 | 15341 | 15745 | 16148 | 16654 |
| 7 | 1 | 8501 | 8752 | 9009 | 9266 | 9536 | 9806 | 10081 | 10355 | 10637 | 10919 | 11203 |
| | 2 | 13748 | 14165 | 14590 | 15015 | 15460 | 15885 | 16358 | 16810 | 17284 | 17758 | 18234 |
| | 3 | 16474 | 16964 | 17466 | 17968 | 18475 | 18981 | 19508 | 20015 | 20545 | 21024 | 21606 |
| 8 | 1 | 9740 | 10030 | 10326 | 10622 | 10931 | 11239 | 11557 | 11874 | 12200 | 12526 | 12856 |
| | 2 | 16225 | 16720 | 17227 | 17733 | 18252 | 18770 | 19311 | 19852 | 20407 | 20961 | 21518 |
| | 3 | 20206 | 20812 | 21432 | 22051 | 22685 | 23318 | 23960 | 24601 | 25261 | 25920 | 26585 |
| | 4 | 21022 | 21638 | 22268 | 22898 | 23549 | 24200 | 24860 | 25531 | 26216 | 26901 | 27590 |
| 9 | 1 | 10961 | 11288 | 11625 | 11961 | 12310 | 12658 | 13018 | 13378 | 13747 | 14116 | 14490 |
| | 2 | 18672 | 19244 | 19832 | 20419 | 21019 | 21618 | 22239 | 22860 | 23503 | 24146 | 24795 |
| | 3 | 23886 | 24603 | 25343 | 26083 | 26839 | 27595 | 28365 | 29135 | 29923 | 30710 | 31500 |
| | 4 | 25943 | 26715 | 27498 | 28281 | 29096 | 29910 | 30741 | 31572 | 32431 | 33290 | 34155 |

## TABLE 9

### Maximum Shears for Truss Bridges—Cooper's *E*50 for One Rail

Shears Given in Thousands of Pounds

Panels 1 2 3 4 5 6 7 8 9

| Panels in Truss | Panel | 8′ 0″ | 8′ 6″ | 9′ 0″ | 9′ 6″ | 10′ 0″ | 10′ 6″ | 11′ 0″ | 11′ 6″ | 12′ 0″ | 12′ 6″ | 13′ 0″ | 13′ 6″ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Panel Lengths | | | | | | | | | | | |
| 3 | 1 | 40.6 | 42.1 | 43.5 | 44.8 | 46.4 | 47.9 | 49.1 | 50.4 | 51.6 | 53.0 | 54.3 | 55.9 |
| | 2 | 7.3 | 8.0 | 8.8 | 9.5 | 10.0 | 11.0 | 11.8 | 12.5 | 13.2 | 13.7 | 14.3 | 14.9 |
| 4 | 1 | 54.1 | 56.7 | 59.1 | 61.3 | 63.1 | 65.5 | 67.4 | 69.4 | 71.6 | 73.6 | 75.5 | 77.6 |
| | 2 | 23.5 | 25.4 | 27.4 | 28.6 | 30.0 | 31.3 | 32.4 | 33.4 | 34.4 | 35.6 | 36.7 | 37.7 |
| | 3 | 2.4 | 3.1 | 3.9 | 4.5 | 5.0 | 5.9 | 6.5 | 7.2 | 7.9 | 8.4 | 8.9 | 9.4 |
| 5 | 1 | 67.5 | 70.4 | 73.6 | 76.6 | 79.4 | 82.3 | 84.5 | 87.1 | 89.2 | 91.4 | 93.6 | 96.4 |
| | 2 | 38.8 | 41.0 | 43.0 | 44.9 | 46.7 | 48.7 | 50.3 | 51.9 | 53.8 | 55.5 | 57.1 | 58.7 |
| | 3 | 16.3 | 18.0 | 19.5 | 20.8 | 22.0 | 23.1 | 24.0 | 25.0 | 25.9 | 26.9 | 27.8 | 28.7 |
| 6 | 1 | 80.1 | 83.5 | 86.9 | 90.1 | 93.6 | 96.9 | 100.1 | 103.1 | 106.7 | 110.5 | 114.3 | 118.7 |
| | 2 | 52.7 | 55.3 | 57.9 | 60.5 | 62.9 | 65.5 | 67.8 | 70.1 | 72.1 | 74.2 | 76.3 | 78.1 |
| | 3 | 30.2 | 33.5 | 34.0 | 35.6 | 37.4 | 39.0 | 40.8 | 41.9 | 43.4 | 44.9 | 46.3 | 47.7 |
| | 4 | 11.5 | 13.0 | 14.4 | 15.6 | 16.6 | 17.8 | 18.8 | 19.4 | 20.2 | 21.1 | 21.9 | 22.6 |
| 7 | 1 | 91.1 | 94.6 | 99.2 | 103.4 | 108.0 | 112.8 | 117 5 | 122.9 | 127.5 | 132.0 | 136.5 | 141.4 |
| | 2 | 65.5 | 69.1 | 72.4 | 75 3 | 78.4 | 80.9 | 83.9 | 86.1 | 89.0 | 92.0 | 95.0 | 98.8 |
| | 3 | 43.4 | 45.6 | 48.0 | 50.4 | 52.4 | 54.8 | 56.9 | 58.8 | 59.6 | 62.0 | 64.3 | 65.9 |
| | 4 | 24.1 | 26.0 | 27.6 | 29.0 | 30.5 | 32.1 | 33.4 | 34.7 | 36.1 | 37.4 | 38.6 | 39.8 |
| | 5 | 8.5 | 9.6 | 10.7 | 11.7 | 12.8 | 13.8 | 14.9 | 15.5 | 16.1 | 16.9 | 17.7 | 18.4 |
| 8 | 1 | 101.9 | 107.6 | 113.6 | 119.3 | 125.4 | 131.0 | 136.4 | 141.9 | 147.2 | 152.3 | 157.4 | 162.9 |
| | 2 | 78.2 | 81.7 | 85.2 | 89.1 | 92.5 | 96.0 | 99.8 | 104.1 | 108.4 | 112.6 | 116.7 | 121.0 |
| | 3 | 55.8 | 59.0 | 61.9 | 64.5 | 67.4 | 69.6 | 72.3 | 74.4 | 76.8 | 79.5 | 82.2 | 85.0 |
| | 4 | 36.4 | 38.5 | 40.6 | 42.8 | 44.6 | 46.8 | 48.6 | 50.4 | 52.0 | 53.7 | 55.3 | 56.7 |
| | 5 | 19.5 | 21.3 | 22.8 | 24.1 | 25.5 | 26.9 | 28.0 | 29.1 | 30.5 | 31.7 | 32.8 | 33.9 |
| | 6 | 7.4 | 7.9 | 8.4 | 9.2 | 10.0 | 10.9 | 11.9 | 12.5 | 13.1 | 13.8 | 14.5 | 15.1 |
| 9 | 1 | 115.2 | 122.3 | 129.2 | 135.6 | 141.9 | 148.4 | 154.5 | 160.8 | 166 4 | 172.0 | 177.6 | 183.5 |
| | 2 | 89.0 | 93.6 | 98.3 | 103.3 | 108.3 | 113.6 | 118.6 | 123.4 | 128.2 | 132.9 | 137.5 | 142.5 |
| | 3 | 68.1 | 71.4 | 74.5 | 77.6 | 81.2 | 84.3 | 87.8 | 91.6 | 95.4 | 99.2 | 102.9 | 106.4 |
| | 4 | 48.2 | 51.1 | 53.8 | 56.5 | 58.5 | 60.8 | 63.1 | 65.1 | 67.4 | 69.8 | 72.2 | 74.8 |
| | 5 | 31.0 | 32.9 | 34.9 | 36.9 | 38.5 | 40.5 | 42.3 | 43.8 | 45.3 | 46.8 | 48.3 | 49.6 |
| | 6 | 16.0 | 17.5 | 19.1 | 20.3 | 21.5 | 22.7 | 23.9 | 25.0 | 26.2 | 27.3 | 28.3 | 29.3 |

| Panels in Truss | Panel | 14′ 0″ | 14′ 6″ | 15′ 0″ | 15′ 6″ | 16′ 0″ | 16′ 6″ | 17′ 0″ | 17′ 6″ | 18′ 0″ | 18′ 6″ | 19′ 0″ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Panel Lengths | | | | | | | | | | |
| 3 | 1 | 57.4 | 58.7 | 60.0 | 61.5 | 63.0 | 64.3 | 65.6 | 66.9 | 68.2 | 69.5 | 70.8 |
| | 2 | 15.5 | 16.0 | 16.4 | 17.1 | 17.8 | 18.3 | 18.8 | 19.3 | 19.9 | 20.5 | 21.0 |
| 4 | 1 | 79.6 | 81.6 | 83.6 | 85.5 | 87.3 | 89.0 | 90.6 | 92.6 | 94.5 | 96.4 | 98.3 |
| | 2 | 38.6 | 39.6 | 40.6 | 41.7 | 42.7 | 43.9 | 45.0 | 46.1 | 47.2 | 48.3 | 49.3 |
| | 3 | 9.8 | 10.3 | 10.7 | 11.2 | 11.7 | 12.2 | 12.7 | 13.1 | 13.5 | 13.9 | 14.3 |
| 5 | 1 | 99.2 | 102.3 | 105.4 | 108.6 | 111.8 | 115.1 | 118.3 | 121.5 | 124.6 | 127.5 | 130.4 |
| | 2 | 60.3 | 61.9 | 63.4 | 64.8 | 66.2 | 67.7 | 69.1 | 70.8 | 72.4 | 74.0 | 75.6 |
| | 3 | 29.5 | 30.4 | 31.2 | 32.0 | 32 8 | 33.6 | 34.3 | 35 1 | 35.8 | 36.6 | 37.3 |
| 6 | 1 | 123.1 | 127.1 | 131.0 | 134.9 | 138.8 | 142.7 | 146.5 | 150 2 | 153.8 | 157.5 | 161.1 |
| | 2 | 79.8 | 82.2 | 84.6 | 86.9 | 90.1 | 93.0 | 95.8 | 98.5 | 101.1 | 103.6 | 106.1 |
| | 3 | 49.1 | 50.4 | 51.7 | 52.9 | 54.0 | 55.3 | 56.5 | 57.6 | 58.6 | 59.7 | 60.7 |
| | 4 | 23.3 | 24.1 | 24.8 | 25.6 | 26.3 | 27.0 | 27.6 | 28.3 | 28.9 | 29.6 | 30.2 |
| 7 | 1 | 146.2 | 150.9 | 155.5 | 160.1 | 164.6 | 169.0 | 173.3 | 177.5 | 181.6 | 185.7 | 189.7 |
| | 2 | 102.6 | 106.1 | 109.6 | 113.0 | 116.4 | 119.7 | 123.1 | 126.4 | 129.6 | 132.8 | 135.9 |
| | 3 | 67.4 | 69.3 | 71.1 | 73.1 | 75.0 | 77.4 | 79.7 | 82.1 | 84.4 | 86.6 | 88.8 |
| | 4 | 41.0 | 42.2 | 43.4 | 44.4 | 45.4 | 46.5 | 47.5 | 48.5 | 49.4 | 50.4 | 51.3 |
| | 5 | 19.0 | 19.7 | 20 3 | 21.0 | 21.6 | 22.2 | 22.8 | 23.4 | 24.0 | 24.6 | 25.1 |
| 8 | 1 | 168.4 | 173.6 | 178.8 | 183.8 | 188.7 | 193.6 | 198.4 | 203.1 | 207.8 | 212.5 | 217.1 |
| | 2 | 125.3 | 129.5 | 133.7 | 137.8 | 141.8 | 145.7 | 149.5 | 153.2 | 156.9 | 160.5 | 164.1 |
| | 3 | 87.8 | 90.9 | 93.9 | 96.8 | 99.6 | 102.6 | 105.6 | 108.5 | 111.4 | 114.2 | 117.0 |
| | 4 | 58.1 | 59.8 | 61.4 | 63.1 | 64.8 | 66.7 | 68.5 | 70.4 | 72.2 | 74.0 | 75.8 |
| | 5 | 35.0 | 36.1 | 37.1 | 38.0 | 38.9 | 39.9 | 40.9 | 41.7 | 42.5 | 43.4 | 44.2 |
| | 6 | 15.7 | 16.4 | 17.0 | 17.6 | 18.1 | 18.7 | 19.2 | 19.8 | 20.3 | 20.8 | 21.3 |
| 9 | 1 | 189.4 | 195.1 | 200.8 | 206.3 | 211.8 | 217.3 | 222.7 | 228 0 | 233.2 | 238.4 | 243.6 |
| | 2 | 147.4 | 152.1 | 156.8 | 161 3 | 165.7 | 170.1 | 174.5 | 178.8 | 183.0 | 187.2 | 191.3 |
| | 3 | 109.8 | 112.9 | 116.7 | 120.4 | 124.1 | 127.6 | 131.0 | 134.4 | 137.7 | 141.0 | 144.2 |
| | 4 | 77.3 | 80.1 | 82.7 | 85.2 | 87.6 | 90.1 | 92.5 | 94.9 | 97.3 | 99.9 | 102.4 |
| | 5 | 50.8 | 52.4 | 53.8 | 55.4 | 56.9 | 58.6 | 60.2 | 61.9 | 63.5 | 65.3 | 67.0 |
| | 6 | 30.3 | 31.4 | 32.3 | 33.1 | 33.9 | 34.8 | 35.7 | 36.5 | 37.2 | 38.0 | 38.7 |

## TABLE 9.—*Continued*

### Maximum Shears for Truss Bridges—Cooper's *E*50 for One Rail
### Shears Given in Thousands of Pounds

Panels 1 2 3 4 5 6 7 8 9

| Panels in Truss | Panel | Panel Lengths | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 19′ 6″ | 20′ 0″ | 20′ 6″ | 21′ 0″ | 21′ 6″ | 22′ 0″ | 22′ 6″ | 23′ 0″ | 23′ 6″ | 24′ 0″ | 24′ 6″ |
| 3 | 1 | 72.0 | 73.3 | 74.3 | 75.3 | 76.6 | 78.0 | 79.5 | 81.0 | 82.1 | 83.2 | 84.6 |
| | 2 | 21.5 | 22.0 | 22.4 | 22.9 | 23.5 | 24.0 | 24.3 | 24.6 | 25.1 | 25.5 | 25.9 |
| 4 | 1 | 100.7 | 103.0 | 105.6 | 108.2 | 110.7 | 113.2 | 115.5 | 117.7 | 120.0 | 122.2 | 124.4 |
| | 2 | 50.3 | 51.3 | 52.2 | 53.1 | 54.0 | 54.9 | 55.8 | 56.8 | 57.4 | 58.2 | 59.0 |
| | 3 | 14.7 | 15.0 | 15.3 | 15.6 | 15.9 | 16.2 | 16.5 | 16.7 | 17.0 | 17.2 | 17.5 |
| 5 | 1 | 133.5 | 136.6 | 139.8 | 142.9 | 146.0 | 149.0 | 152.0 | 154.9 | 157.8 | 160.5 | 163.3 |
| | 2 | 77.4 | 79.1 | 80.9 | 82.6 | 84.4 | 86.1 | 88.0 | 89.9 | 91.7 | 93.5 | 95.1 |
| | 3 | 38.1 | 38.8 | 39.6 | 40.3 | 40.9 | 41.6 | 42.3 | 42.9 | 43.7 | 44.3 | 45.0 |
| 6 | 1 | 164.6 | 168.1 | 171.7 | 175.2 | 178.8 | 182.3 | 185.8 | 189.2 | 192.6 | 195.9 | 199.2 |
| | 2 | 108.6 | 111.0 | 113.6 | 116.0 | 118.5 | 120.8 | 123.2 | 125.4 | 127.9 | 130.1 | 132.4 |
| | 3 | 62.1 | 63.5 | 65.1 | 66.6 | 68.2 | 69.6 | 71.3 | 72.9 | 74.5 | 75.9 | 77.4 |
| | 4 | 30.8 | 31.4 | 32.1 | 32.8 | 33.4 | 34.0 | 34.5 | 35.0 | 35.5 | 36.0 | 36.6 |
| 7 | 1 | 193.9 | 197.8 | 201.7 | 205.5 | 209.6 | 213.7 | 217.8 | 221.8 | 225.8 | 229.7 | 233.6 |
| | 2 | 139.0 | 142.0 | 145.0 | 147.9 | 150.9 | 153.7 | 156.1 | 159.3 | 162.1 | 164.8 | 167.6 |
| | 3 | 91.0 | 93.1 | 95.4 | 97.5 | 99.6 | 101.6 | 103.8 | 105.8 | 107.9 | 109.8 | 111.8 |
| | 4 | 52.4 | 53.4 | 54.5 | 55.5 | 56.7 | 57.8 | 59.3 | 60.6 | 62.1 | 63.4 | 64.7 |
| | 5 | 25.7 | 26.3 | 26.9 | 27.4 | 28.0 | 28.5 | 29.0 | 29.4 | 29.9 | 30.3 | 30.8 |
| 8 | 1 | 221.7 | 226.3 | 230.8 | 235.2 | 239.8 | 244.3 | 248.9 | 253.4 | 258.0 | 262.5 | 267.1 |
| | 2 | 167.7 | 171.3 | 174.8 | 178.2 | 181.7 | 185.0 | 188.4 | 191.7 | 195.1 | 198.3 | 201.7 |
| | 3 | 119.8 | 122.5 | 125.1 | 127.6 | 130.5 | 132.8 | 135.4 | 137.8 | 140.3 | 142.7 | 145.2 |
| | 4 | 77.8 | 79.8 | 81.7 | 83.6 | 85.5 | 87.3 | 89.2 | 91.0 | 92.8 | 94.5 | 96.3 |
| | 5 | 45.2 | 46.1 | 47.1 | 48.0 | 49.0 | 49.4 | 51.0 | 52.1 | 53.1 | 54.1 | 55.3 |
| | 6 | 21.9 | 22.4 | 22.9 | 23.4 | 23.9 | 24.4 | 24.9 | 25.3 | 25.7 | 26.0 | 26.5 |
| 9 | 1 | 248.8 | 253.9 | 259.0 | 264.0 | 269.2 | 274.2 | 279.4 | 284.5 | 289.7 | 294.8 | 299.9 |
| | 2 | 195.4 | 199.5 | 203.5 | 207.5 | 211.5 | 215.5 | 219.4 | 223.3 | 227.2 | 231.0 | 234.9 |
| | 3 | 147.4 | 150.6 | 153.8 | 156.9 | 160.0 | 163.0 | 166.0 | 169.0 | 172.0 | 175.0 | 177.9 |
| | 4 | 104.9 | 107.3 | 109.7 | 112.0 | 114.3 | 116.6 | 118.9 | 121.1 | 123.4 | 125.5 | 127.8 |
| | 5 | 68.6 | 70.1 | 71.7 | 73.3 | 74.9 | 76.4 | 78.0 | 79.5 | 81.2 | 82.8 | 84.3 |
| | 6 | 39.6 | 40.4 | 41.3 | 42.1 | 43.0 | 43.9 | 44.9 | 45.8 | 46.7 | 47.6 | 48.6 |

| Panels in Truss | Panel | Panel Lengths | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 25′ 0″ | 25′ 6″ | 26′ 0″ | 26′ 6″ | 27′ 0″ | 27′ 6″ | 28′ 0″ | 28′ 6″ | 29′ 0″ | 29′ 6″ | 30′ 0″ |
| 3 | 1 | 86.0 | 87.0 | 88.0 | 89.5 | 91.0 | 92.2 | 93.5 | 94.7 | 96.0 | 97.8 | 99.7 |
| | 2 | 26.4 | 26.8 | 27.2 | 27.6 | 28.0 | 28.3 | 28.6 | 29.0 | 29.4 | 29.7 | 30.0 |
| 4 | 1 | 126.5 | 128.7 | 130.9 | 133.1 | 135.2 | 137.3 | 139.3 | 141.5 | 143.6 | 145.8 | 147.9 |
| | 2 | 59.7 | 60.5 | 61.3 | 62.1 | 62.9 | 63.8 | 64.6 | 65.6 | 66.5 | 67.4 | 68.3 |
| | 3 | 17.8 | 18.1 | 18.4 | 18.6 | 18.9 | 19.1 | 19.3 | 19.6 | 19.8 | 20.1 | 20.3 |
| 5 | 1 | 166.0 | 168.8 | 171.4 | 174.1 | 176.7 | 179.4 | 181.9 | 184.5 | 187.0 | 189.6 | 192.0 |
| | 2 | 96.6 | 98.3 | 100.1 | 101.9 | 103.6 | 105.4 | 107.1 | 108.9 | 110.6 | 112.3 | 114.0 |
| | 3 | 45.5 | 46.3 | 46.9 | 47.7 | 48.3 | 49.0 | 49.6 | 50.5 | 51.3 | 52.1 | 52.8 |
| 6 | 1 | 202.5 | 205.8 | 209.0 | 212.2 | 215.4 | 218.6 | 221.8 | 224.9 | 228.0 | 231.1 | 234.2 |
| | 2 | 134.5 | 136.8 | 139.0 | 141.3 | 143.5 | 145.8 | 148.0 | 150.3 | 152.4 | 154.6 | 156.7 |
| | 3 | 78.6 | 80.2 | 81.5 | 83.0 | 84.3 | 85.7 | 87.0 | 88.4 | 89.6 | 91.1 | 92.4 |
| | 4 | 37.1 | 37.6 | 38.1 | 38.6 | 39.1 | 39.6 | 40.0 | 40.5 | 41.0 | 41.7 | 42.4 |
| 7 | 1 | 237.4 | 241.4 | 245.2 | 249.1 | 252.8 | 256.6 | 260.3 | 264.1 | 267.7 | 271.4 | 275.0 |
| | 2 | 170.3 | 173.2 | 175.9 | 178.8 | 181.5 | 184.3 | 187.0 | 189.8 | 192.5 | 195.3 | 197.9 |
| | 3 | 113.6 | 115.6 | 117.4 | 119.3 | 121.1 | 123.0 | 124.8 | 126.6 | 128.3 | 130.2 | 131.9 |
| | 4 | 65.8 | 67.1 | 68.3 | 69.6 | 70.8 | 72.0 | 73.1 | 74.3 | 75.4 | 76.7 | 77.8 |
| | 5 | 31.3 | 31.8 | 32.1 | 32.6 | 33.0 | 33.5 | 33.8 | 34.3 | 34.6 | 35.1 | 35.6 |
| 8 | 1 | 271.5 | 276.0 | 280.4 | 284.9 | 289.2 | 293.6 | 297.9 | 302.3 | 306.5 | 310.8 | 315.0 |
| | 2 | 204.9 | 208.3 | 211.6 | 215.1 | 218.4 | 221.8 | 225.0 | 228.4 | 231.7 | 235.0 | 238.2 |
| | 3 | 147.5 | 150.0 | 152.3 | 154.7 | 157.0 | 159.4 | 161.7 | 164.0 | 166.1 | 168.5 | 170.2 |
| | 4 | 98.0 | 99.8 | 101.4 | 103.1 | 104.6 | 106.3 | 107.9 | 109.5 | 111.0 | 112.6 | 114.1 |
| | 5 | 56.4 | 57.4 | 58.4 | 59.5 | 60.5 | 61.6 | 62.6 | 63.7 | 64.8 | 65.9 | 66.9 |
| | 6 | 26.9 | 27.3 | 27.6 | 28.0 | 28.4 | 28.8 | 29.1 | 29.5 | 29.9 | 30.4 | 30.8 |
| 9 | 1 | 304.9 | 310.0 | 315.0 | 320.1 | 325.0 | 330.0 | 334.9 | 339.9 | 344.7 | 349.7 | 354.5 |
| | 2 | 238.8 | 242.8 | 246.7 | 250.6 | 254.5 | 258.5 | 262.4 | 266.3 | 270.2 | 274.0 | 277.8 |
| | 3 | 180.8 | 183.8 | 186.7 | 189.6 | 192.4 | 195.3 | 198.0 | 200.9 | 203.8 | 206.7 | 209.5 |
| | 4 | 129.9 | 132.0 | 134.1 | 136.3 | 138.4 | 140.5 | 142.5 | 144.6 | 146.6 | 148.6 | 150.6 |
| | 5 | 85.8 | 87.4 | 88.9 | 90.4 | 91.8 | 93.3 | 94.8 | 96.2 | 97.6 | 99.0 | 100.4 |
| | 6 | 49.6 | 50.6 | 51.5 | 52.4 | 53.3 | 54.2 | 55.0 | 55.9 | 56.8 | 57.6 | 58.4 |

TABLE 9.—*Continued*

MAXIMUM SHEARS FOR TRUSS BRIDGES—COOPER'S *E*50 FOR ONE RAIL
Shears Given in Thousands of Pounds

Panels 1 2 3 4 5 6 7 8 9

| Panels in Truss | Panel | Panel Lengths | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 30′ 6″ | 31′ 0″ | 31′ 6″ | 32′ 0″ | 32′ 6″ | 33′ 0″ | 33′ 6″ | 34′ 0″ | 34′ 6″ | 35′ 0″ | 35′ 6″ |
| 3 | 1 | 101.1 | 102.6 | 104.6 | 106.6 | 108.1 | 109.6 | 111.5 | 113.4 | 114.8 | 116.2 | 117.6 |
| | 2 | 30.4 | 30.8 | 31.2 | 31.5 | 31.8 | 32.2 | 32.5 | 32.8 | 33.1 | 33.4 | 33.7 |
| 4 | 1 | 149.9 | 152.0 | 154.0 | 156.1 | 158.0 | 160.0 | 161.9 | 163.8 | 165.8 | 167.9 | 169.8 |
| | 2 | 69.1 | 70.0 | 71.7 | 73.3 | 74.4 | 75.4 | 76.4 | 77.4 | 78.4 | 79.4 | 80.5 |
| | 3 | 20.6 | 20.9 | 21.1 | 21.3 | 21.6 | 22.0 | 22.2 | 22.5 | 22.7 | 23.0 | 23.3 |
| 5 | 1 | 194.6 | 197.1 | 199.8 | 202.4 | 205.0 | 207.5 | 210.1 | 212.6 | 215.1 | 217.6 | 220.2 |
| | 2 | 115.6 | 117.3 | 118.9 | 120.4 | 122.0 | 123.5 | 125.0 | 126.5 | 128.0 | 129.5 | 131.0 |
| | 3 | 53.6 | 54.3 | 55.1 | 55.9 | 56.7 | 57.4 | 58.3 | 59.1 | 60.0 | 60.8 | 61.7 |
| 6 | 1 | 237.3 | 240.3 | 243.5 | 246.6 | 249.8 | 252.9 | 256.0 | 259.1 | 262.3 | 265.4 | 268.5 |
| | 2 | 158.8 | 160.9 | 163.0 | 165.1 | 167.2 | 169.3 | 171.4 | 173.4 | 175.4 | 177.4 | 179.4 |
| | 3 | 93.7 | 95.0 | 96.3 | 97.5 | 98.8 | 100.0 | 101.3 | 102.5 | 103.8 | 105.1 | 106.4 |
| | 4 | 43.0 | 43.6 | 44.4 | 45.1 | 45.8 | 46.4 | 47.2 | 47.9 | 48.6 | 49.3 | 50.0 |
| 7 | 1 | 278.7 | 282.3 | 286.0 | 289.6 | 293.4 | 297.1 | 300.9 | 304.7 | 308.4 | 312.0 | 315.7 |
| | 2 | 200.6 | 203.3 | 205.9 | 208.5 | 211.2 | 213.8 | 216.4 | 218.9 | 221.5 | 224.0 | 226.5 |
| | 3 | 133.6 | 135.3 | 137.1 | 138.9 | 140.7 | 142.5 | 144.3 | 146.0 | 147.9 | 149.8 | 151.7 |
| | 4 | 79.0 | 80.1 | 81.3 | 82.4 | 83.5 | 84.5 | 85.6 | 86.6 | 87.7 | 88.7 | 89.8 |
| | 5 | 36.1 | 36.5 | 37.0 | 37.5 | 38.0 | 38.5 | 39.2 | 39.9 | 40.5 | 41.0 | 41.6 |
| 8 | 1 | 319.3 | 323.5 | 327.8 | 332.0 | 337.0 | 341.9 | 345.6 | 349.3 | 353.2 | 357.0 | 360.9 |
| | 2 | 241.4 | 244.6 | 247.8 | 251.0 | 254.2 | 257.4 | 260.6 | 263.8 | 266.9 | 270.0 | 273.2 |
| | 3 | 172.8 | 175.4 | 177.8 | 180.1 | 182.5 | 184.8 | 187.1 | 189.4 | 191.7 | 193.9 | 196.2 |
| | 4 | 115.7 | 117.3 | 118.7 | 120.3 | 121.9 | 123.4 | 124.9 | 126.3 | 127.7 | 129.1 | 130.5 |
| | 5 | 67.9 | 68.9 | 69.9 | 70.9 | 71.9 | 72.9 | 73.9 | 74.8 | 75.7 | 76.6 | 77.5 |
| | 6 | 31.2 | 31.5 | 32.0 | 32.5 | 32.9 | 33.3 | 33.8 | 34.3 | 34.7 | 35.1 | 35.5 |
| 9 | 1 | 359.4 | 364.2 | 369.1 | 373.9 | 378.7 | 383.5 | 388.5 | 393.5 | 398.4 | 403.3 | 408.3 |
| | 2 | 281.6 | 285.4 | 289.2 | 293.0 | 296.8 | 300.5 | 304.3 | 308.0 | 311.8 | 315.5 | 319.2 |
| | 3 | 212.4 | 215.3 | 218.2 | 221.0 | 223.9 | 226.8 | 229.6 | 232.5 | 235.3 | 238.1 | 240.8 |
| | 4 | 152.7 | 154.8 | 156.8 | 158.8 | 160.7 | 162.6 | 164.6 | 166.6 | 168.6 | 170.5 | 172.5 |
| | 5 | 101.8 | 103.1 | 104.5 | 105.9 | 107.3 | 108.6 | 110.0 | 111.4 | 112.7 | 114.0 | 115.4 |
| | 6 | 59.4 | 60.3 | 61.2 | 62.0 | 62.9 | 63.8 | 64.7 | 65.5 | 66.3 | 67.1 | 67.8 |

## TABLE 10

### Maximum Bending Moments in Girder Bridges Without Floor-Beams, Cooper's $E$40 Loading

Values in Thousands of Foot-Pounds per Rail

| Longer Segment $l_2$ | Shorter Segment $l_1$: 5 | 10 | 15 | 20 | 25 | 30 | 35 | 40 | 45 | 50 | 55 | 60 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 250.. | 1534 | 3030 | 4514 | 5979 | 7411 | 8820 | 10203 | 11562 | 12916 | 14278 | 15628 | 16982 |
| 225.. | 1404 | 2769 | 4122 | 5455 | 6758 | 8034 | 9288 | 10515 | 11743 | 12976 | 14198 | 15422 |
| 200.. | 1273 | 2505 | 3727 | 4926 | 6098 | 7241 | 8364 | 9460 | 10560 | 11665 | 12759 | 13849 |
| 175.. | 1139 | 2236 | 3326 | 4390 | 5430 | 6438 | 7430 | 8391 | 9364 | 10339 | 11306 | 12266 |
| 160.. | 1053 | 2073 | 3082 | 4063 | 5022 | 5950 | 6862 | 7742 | 8638 | 9535 | 10424 | 11300 |
| 150.. | 1003 | 1962 | 2917 | 3843 | 4749 | 5620 | 6480 | 7304 | 8150 | 8994 | 9833 | 10664 |
| 140.. | 947 | 1851 | 2750 | 3620 | 4471 | 5287 | 6093 | 6862 | 7658 | 8450 | 9236 | 10016 |
| 130.. | 889 | 1738 | 2582 | 3394 | 4191 | 4951 | 5703 | 6417 | 7161 | 7901 | 8635 | 9363 |
| 120.. | 834 | 1625 | 2410 | 3164 | 3906 | 4608 | 5307 | 5964 | 6658 | 7345 | 8028 | 8704 |
| 110.. | 774 | 1509 | 2234 | 2930 | 3617 | 4260 | 4905 | 5514 | 6148 | 6782 | 7414 | 8038 |
| 100.. | 714 | 1390 | 2055 | 2690 | 3320 | 3910 | 4494 | 5053 | 5650 | 6234 | 6813 | 7387 |
| 95.. | 682 | 1329 | 1963 | 2566 | 3169 | 3730 | 4290 | 4864 | 5431 | 5991 | 6546 | 7096 |
| 90.. | 650 | 1264 | 1866 | 2444 | 3016 | 3550 | 4114 | 4661 | 5202 | 5734 | 6263 | 6786 |
| 85.. | 617 | 1200 | 1770 | 2314 | 2854 | 3365 | 3923 | 4442 | 4936 | 5458 | 5958 | 6449 |
| 80.. | 584 | 1134 | 1671 | 2186 | 2694 | 3200 | 3715 | 4205 | 4690 | 5171 | 5646 | 6117 |
| 75.. | 551 | 1070 | 1573 | 2054 | 2530 | 3008 | 3489 | 3964 | 4422 | 4874 | 5320 | 5761 |
| 70.. | 516 | 1003 | 1474 | 1923 | 2366 | 2805 | 3254 | 3706 | 4132 | 4553 | 4967 | 5378 |
| 65.. | 482 | 931 | 1367 | 1792 | 2202 | 2602 | 3019 | 3437 | 3831 | 4221 | 4608 | 4993 |
| 60.. | 453 | 864 | 1266 | 1649 | 2025 | 2389 | 2770 | 3155 | 3519 | 3884 | 4243 | 4597 |
| 55.. | 425 | 805 | 1172 | 1518 | 1856 | 2195 | 2546 | 2884 | 3214 | 3514 | 3859 | ..... |
| 50.. | 397 | 750 | 1091 | 1398 | 1713 | 2023 | 2336 | 2634 | 2928 | 3219 | ..... | ..... |
| 45.. | 367 | 692 | 1005 | 1290 | 1567 | 1847 | 2136 | 2404 | 2669 | ..... | ..... | ..... |
| 40.. | 335 | 635 | 918 | 1171 | 1419 | 1669 | 1921 | 2160 | ..... | ..... | ..... | ..... |
| 35.. | 302 | 570 | 819 | 1050 | 1272 | 1490 | 1707 | ..... | ..... | ..... | ..... | ..... |
| 30.. | 270 | 506 | 721 | 918 | 1109 | 1294 | ..... | ..... | ..... | ..... | ..... | ..... |
| 25.. | 235 | 440 | 622 | 787 | 946 | .... | ..... | ..... | ..... | ..... | ..... | ..... |
| 20.. | 200 | 373 | 518 | 656 | .... | .... | ..... | ..... | ..... | ..... | ..... | ..... |
| 15.. | 150 | 300 | 410 | .... | .... | .... | ..... | ..... | ..... | ..... | ..... | ..... |
| 10.. | 100 | 200 | .... | .... | .... | .... | ..... | ..... | ..... | ..... | ..... | ..... |
| 5.. | 50 | .... | .... | .... | .... | .... | ..... | ..... | ..... | ..... | ..... | ..... |

For $l_1$ and $l_2$ each > 142 ft. $M = l_1 l_2 + 3800 \frac{l_2}{L}$

TABLE 10.—*Continued*

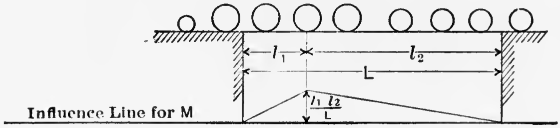
MAXIMUM BENDING MOMENTS IN GIRDER BRIDGES WITHOUT FLOOR-BEAMS, COOPER'S $E40$ LOADING

Values in Thousands of Foot-Pounds per Rail

SHORTER SEGMENT $l_1$

| | 65 | 70 | 75 | 80 | 85 | 90 | 95 | 100 | 110 | 120 | 130 | 140 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 250 | 18327 | 19675 | 21062 | 22421 | 23766 | 25084 | 26364 | 27660 | 30152 | 32591 | 35033 | 37455 |
| 225 | 16639 | 17862 | 19123 | 20351 | 21569 | 22757 | 23908 | 25078 | 27315 | 29502 | 31691 | 33862 |
| 200 | 14939 | 16036 | 17172 | 18269 | 19360 | 20418 | 21440 | 22482 | 24465 | 26400 | 28231 | 30255 |
| 175 | 13224 | 14205 | 15207 | 16171 | 17134 | 18017 | 18952 | 19868 | 21597 | 23278 | 24963 | 26631 |
| 160 | 12185 | 13097 | 14018 | 14906 | 15789 | 16636 | 17450 | 18289 | 19866 | 21396 | 22930 | 24446 |
| 150 | 11487 | 12354 | 13194 | 14058 | 14887 | 15681 | 16442 | 17231 | 18706 | 20151 | 21569 | 22986 |
| 140 | 10790 | 11608 | 12395 | 13206 | 13980 | 14722 | 15430 | 16169 | 17542 | 18870 | 20203 | 21520 |
| 130 | 10088 | 10857 | 11594 | 12349 | 13069 | 13756 | 14413 | 15101 | 16372 | 17600 | 18834 | ..... |
| 120 | 9380 | 10100 | 10786 | 11486 | 12073 | 12787 | 13421 | 14026 | 15197 | 16325 | ..... | ..... |
| 110 | 8666 | 9338 | 9972 | 10616 | 11226 | 11812 | 11392 | 12946 | 14014 | ..... | ..... | ..... |
| 100 | 7963 | 8567 | 9150 | 9738 | 10294 | 10829 | 11348 | 11857 | ..... | ..... | ..... | ..... |
| 95 | 7642 | 8182 | 8737 | 9296 | 9824 | 10334 | 10834 | ..... | ..... | ..... | ..... | ..... |
| 90 | 7303 | 7817 | 8321 | 8851 | 9352 | 9836 | ..... | ..... | ..... | ..... | ..... | ..... |
| 85 | 6943 | 7428 | 7917 | 8404 | 8876 | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 80 | 6582 | 7043 | 7500 | 7954 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 75 | 6197 | 6629 | 7057 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 70 | 5796 | 6197 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 65 | 5374 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |

For $l_1$ and $l_2$ each $>$ 142 ft. $M = l_1 \, l_2 + 3800 \dfrac{l_2}{L}$



## TABLE 11

### Maximum Bending Moments in Girder Bridges Without Floor-Beams, Cooper's, $E50$ Loading

Values in Thousands of Foot-Pounds per Rail

Shorter Segment $l_1$

| Longer Segment $l_2$ | 5 | 10 | 15 | 20 | 25 | 30 | 35 | 40 | 45 | 50 | 55 | 60 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 250. | 1918 | 3788 | 5643 | 7474 | 9264 | 11025 | 12754 | 14452 | 16145 | 17848 | 19535 | 21228 |
| 225. | 1755 | 3461 | 5153 | 6819 | 8447 | 10043 | 11610 | 13144 | 14679 | 16220 | 17748 | 19278 |
| 200. | 1591 | 3131 | 4659 | 6158 | 7622 | 9052 | 10456 | 11825 | 13200 | 14581 | 15949 | 17311 |
| 175. | 1424 | 2795 | 4158 | 5487 | 6787 | 8048 | 9288 | 10489 | 11705 | 12924 | 14132 | 15333 |
| 160. | 1316 | 2591 | 3852 | 5079 | 6278 | 7437 | 8578 | 9677 | 10798 | 11919 | 13030 | 14125 |
| 150. | 1254 | 2453 | 3646 | 4804 | 5936 | 7025 | 8100 | 9130 | 10187 | 11243 | 12291 | 13330 |
| 140. | 1184 | 2314 | 3438 | 4525 | 5589 | 6609 | 7617 | 8578 | 9572 | 10562 | 11545 | 12520 |
| 130. | 1114 | 2173 | 3227 | 4242 | 5239 | 6189 | 7129 | 8021 | 8951 | 9876 | 10794 | 11704 |
| 120. | 1042 | 2031 | 3012 | 3955 | 4883 | 5760 | 6634 | 7455 | 8322 | 9181 | 10035 | 10880 |
| 110. | 968 | 1886 | 2793 | 3662 | 4521 | 5325 | 6131 | 6892 | 7685 | 8478 | 9268 | 10048 |
| 100. | 892 | 1737 | 2569 | 3362 | 4150 | 4887 | 5618 | 6316 | 7063 | 7793 | 8516 | 9234 |
| 95. | 853 | 1661 | 2454 | 3208 | 3961 | 4663 | 5363 | 6080 | 6789 | 7489 | 8183 | 8870 |
| 90. | 812 | 1580 | 2333 | 3055 | 3770 | 4437 | 5143 | 5826 | 6502 | 7168 | 7829 | 8482 |
| 85. | 771 | 1500 | 2213 | 2893 | 3568 | 4206 | 4904 | 5552 | 6170 | 6823 | 7448 | 8061 |
| 80. | 730 | 1418 | 2089 | 2733 | 3368 | 4000 | 4644 | 5256 | 5862 | 6464 | 7058 | 7646 |
| 75. | 689 | 1337 | 1966 | 2568 | 3163 | 3760 | 4361 | 4955 | 5528 | 6093 | 6650 | 7201 |
| 70. | 645 | 1254 | 1843 | 2404 | 2958 | 3506 | 4068 | 4632 | 5165 | 5691 | 6209 | 6723 |
| 65. | 602 | 1164 | 1709 | 2240 | 2753 | 3253 | 3774 | 4296 | 4789 | 5276 | 5760 | 6241 |
| 60. | 566 | 1080 | 1582 | 2061 | 2531 | 2986 | 3463 | 3943 | 4399 | 4855 | 5304 | 5746 |
| 55. | 531 | 1006 | 1465 | 1897 | 2320 | 2744 | 3182 | 3605 | 4017 | 4392 | 4824 | ..... |
| 50. | 496 | 937 | 1364 | 1747 | 2141 | 2529 | 2920 | 3293 | 3660 | 4024 | ..... | ..... |
| 45. | 459 | 865 | 1256 | 1613 | 1959 | 2309 | 2670 | 3005 | 3336 | ..... | ..... | ..... |
| 40. | 419 | 794 | 1147 | 1464 | 1774 | 2086 | 2401 | 2700 | ..... | ..... | ..... | ..... |
| 35. | 377 | 713 | 1024 | 1312 | 1590 | 1862 | 2134 | ..... | ..... | ..... | ..... | ..... |
| 30. | 338 | 632 | 901 | 1148 | 1386 | 1617 | ..... | ..... | ..... | ..... | ..... | ..... |
| 25. | 294 | 550 | 778 | 984 | 1182 | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 20. | 250 | 466 | 647 | 820 | .... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 15. | 187 | 375 | 513 | .... | .... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 10. | 125 | 250 | .... | .... | .... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 5. | 62 | .... | .... | .... | .... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |

For $l_1$ and $l_2$ each $>$ 142 ft. $M = 1.25\, l_1\, l_2 + 4750 \dfrac{l_2}{L}$

## TABLE 11.—*Continued*

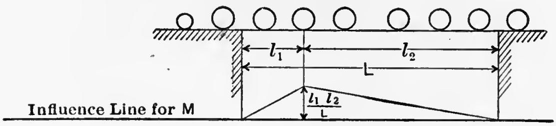
### Maximum Bending Moments in Girder Bridges Without Floor-Beams, Cooper's $E$50 Loading

Values in Thousands of Foot-Pounds per Rail

Shorter Segment $l_1$

| | 65 | 70 | 75 | 80 | 85 | 90 | 95 | 100 | 110 | 120 | 130 | 140 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 250 | 22909 | 24594 | 26327 | 28026 | 29707 | 31355 | 32955 | 34575 | 37690 | 40739 | 43791 | 46819 |
| 225 | 20799 | 22327 | 23904 | 25439 | 26961 | 28446 | 29885 | 31347 | 34144 | 36878 | 39614 | 42327 |
| 200 | 18674 | 20045 | 21465 | 22836 | 24200 | 25522 | 26800 | 28102 | 30581 | 33000 | 35414 | 37819 |
| 175 | 16530 | 17756 | 19009 | 20214 | 21417 | 22521 | 23690 | 24835 | 26996 | 29098 | 31204 | 33289 |
| 160 | 15231 | 16371 | 17523 | 18633 | 19736 | 20795 | 21812 | 22861 | 24832 | 26745 | 28662 | 30558 |
| 150 | 14359 | 15443 | 16492 | 17573 | 18609 | 19601 | 20553 | 21539 | 23382 | 25189 | 26961 | 28732 |
| 140 | 13488 | 14510 | 15494 | 16508 | 17475 | 18402 | 19288 | 20211 | 21927 | 23588 | 25254 | 26900 |
| 130 | 12610 | 13571 | 14492 | 15436 | 16336 | 17195 | 18016 | 18876 | 20465 | 22000 | 23542 | ..... |
| 120 | 11725 | 12625 | 13482 | 14357 | 15091 | 15984 | 16776 | 17533 | 18996 | 20406 | ..... | ..... |
| 110 | 10832 | 11672 | 12465 | 13270 | 14033 | 14765 | 15490 | 16182 | 17518 | ..... | ..... | ..... |
| 100 | 9954 | 10709 | 11438 | 12173 | 12867 | 13536 | 14185 | 14821 | ..... | ..... | ..... | ..... |
| 95 | 9552 | 10227 | 10921 | 11620 | 12280 | 12917 | 13543 | ..... | ..... | ..... | ..... | ..... |
| 90 | 9129 | 9771 | 10401 | 11064 | 11690 | 12295 | ..... | ..... | ..... | ..... | ..... | ..... |
| 85 | 8679 | 9285 | 9896 | 10505 | 11095 | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 80 | 8228 | 8804 | 9375 | 9943 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 75 | 7746 | 8286 | 8821 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 70 | 7237 | 7746 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 65 | 6718 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |

or $l_1$ and $l_2$ each $> 142$ ft. $M = 1.25\, l_1\, l_2 + 4750 \dfrac{l_2}{L}$



## TABLE 12

### MAXIMUM BENDING MOMENTS IN GIRDER BRIDGES WITHOUT FLOOR-BEAMS, COOPER'S *E*60 LOADING

Values in Thousands of Foot-pounds per Rail

SHORTER SEGMENT $l_1$

| Longer Segment $l_2$ | 5 | 10 | 15 | 20 | 25 | 30 | 35 | 40 | 45 | 50 | 55 | 60 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 250 | 2302 | 4547 | 6772 | 8969 | 11117 | 13230 | 15305 | 17342 | 19374 | 21418 | 23442 | 25474 |
| 225 | 2106 | 4153 | 6184 | 8183 | 10136 | 12052 | 13932 | 15773 | 17615 | 19464 | 21298 | 23134 |
| 200 | 1909 | 3757 | 5591 | 7390 | 9146 | 10862 | 12547 | 14190 | 15840 | 17497 | 19139 | 20773 |
| 175 | 1709 | 3354 | 4990 | 6584 | 8144 | 9658 | 11146 | 12587 | 14046 | 15509 | 16958 | 18400 |
| 160 | 1579 | 3109 | 4622 | 6095 | 7534 | 8924 | 10294 | 11612 | 12958 | 14303 | 15636 | 16950 |
| 150 | 1505 | 2944 | 4375 | 5765 | 7123 | 8430 | 9720 | 10956 | 12224 | 13492 | 14749 | 15996 |
| 140 | 1421 | 2777 | 4126 | 5430 | 6707 | 7931 | 9140 | 10294 | 11486 | 12674 | 13854 | 15024 |
| 130 | 1337 | 2608 | 3872 | 5090 | 6287 | 7427 | 8555 | 9625 | 10741 | 11851 | 12953 | 14045 |
| 120 | 1250 | 2437 | 3614 | 4746 | 5860 | 6912 | 7961 | 8946 | 9986 | 11017 | 12042 | 13056 |
| 110 | 1162 | 2263 | 3352 | 4394 | 5425 | 6390 | 7357 | 8270 | 9222 | 10174 | 11122 | 12058 |
| 100 | 1070 | 2084 | 3083 | 4034 | 4980 | 5864 | 6742 | 7579 | 8476 | 9352 | 10219 | 11081 |
| 95 | 1024 | 1993 | 2945 | 3850 | 4753 | 5596 | 6436 | 7296 | 8147 | 8987 | 9820 | 10644 |
| 90 | 974 | 1896 | 2800 | 3666 | 4524 | 5324 | 6172 | 6991 | 7802 | 8602 | 9395 | 10178 |
| 85 | 925 | 1800 | 2656 | 3472 | 4282 | 5047 | 5885 | 6662 | 7404 | 8188 | 8938 | 9673 |
| 80 | 876 | 1702 | 2507 | 3280 | 4042 | 4800 | 5573 | 6307 | 7034 | 7757 | 8470 | 9175 |
| 75 | 827 | 1604 | 2359 | 3082 | 3796 | 4512 | 5233 | 5946 | 6634 | 7312 | 7980 | 8641 |
| 70 | 774 | 1505 | 2212 | 2885 | 3550 | 4207 | 4882 | 5558 | 6198 | 6829 | 7451 | 8068 |
| 65 | 722 | 1397 | 2051 | 2688 | 3304 | 3903 | 4529 | 5155 | 5747 | 6331 | 6912 | 7489 |
| 60 | 679 | 1296 | 1898 | 2473 | 3037 | 3583 | 4156 | 4732 | 5279 | 5826 | 6365 | 6895 |
| 55 | 637 | 1207 | 1758 | 2276 | 2784 | 3293 | 3818 | 4326 | 4820 | 5270 | 5789 | .... |
| 50 | 595 | 1124 | 1637 | 2096 | 2569 | 3035 | 3504 | 3952 | 4392 | 4829 | .... | .... |
| 45 | 551 | 1038 | 1507 | 1936 | 2351 | 2771 | 3204 | 3606 | 4003 | .... | .... | .... |
| 40 | 503 | 953 | 1376 | 1757 | 2129 | 2503 | 2881 | 3240 | .... | .... | .... | .... |
| 35 | 452 | 856 | 1229 | 1574 | 1908 | 2234 | 2561 | .... | .... | .... | .... | .... |
| 30 | 406 | 758 | 1081 | 1378 | 1663 | 1940 | .... | .... | .... | .... | .... | .... |
| 25 | 353 | 660 | 934 | 1181 | 1418 | .... | .... | .... | .... | .... | .... | .... |
| 20 | 300 | 559 | 776 | 984 | .... | .... | .... | .... | .... | .... | .... | .... |
| 15 | 224 | 450 | 616 | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| 10 | 150 | 300 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| 5 | 74 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |

For $l_1$ and $l_2$ each $>$ 142 ft. $M = 1.5\, l_1 l_2 + 5700 \dfrac{l_2}{L}$

## TABLE 12.—*Continued*

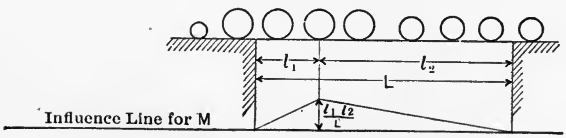
### Maximum Bending Moments in Girder Bridges Without Floor-Beams, Cooper's $E60$ Loading

Values in Thousands of Foot-pounds per Rail

Shorter Segment $l_1$

| | 65 | 70 | 75 | 80 | 85 | 90 | 95 | 100 | 110 | 120 | 130 | 140 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 250 | 27491 | 29513 | 31592 | 33631 | 35648 | 37626 | 39546 | 41490 | 45228 | 48887 | 52549 | 56183 |
| 225 | 24959 | 26792 | 28685 | 30527 | 32353 | 34135 | 35862 | 37616 | 40973 | 44254 | 47537 | 50792 |
| 200 | 22409 | 24054 | 25758 | 27403 | 29040 | 30626 | 32160 | 33722 | 36697 | 39600 | 42497 | 45383 |
| 175 | 19836 | 21307 | 22811 | 24257 | 25700 | 27025 | 28428 | 29802 | 32395 | 34918 | 37444 | 39947 |
| 160 | 18277 | 19645 | 21028 | 22360 | 23683 | 24954 | 26174 | 27433 | 29798 | 32094 | 34394 | 36670 |
| 150 | 17231 | 18532 | 19790 | 21088 | 22331 | 23521 | 24664 | 25847 | 28058 | 30227 | 32353 | 34478 |
| 40 | 16186 | 17412 | 18593 | 19810 | 20970 | 22082 | 23146 | 24253 | 26312 | 28306 | 30305 | 32280 |
| 130 | 15132 | 16285 | 17390 | 18523 | 19603 | 20634 | 21619 | 22651 | 24558 | 26400 | 28250 | ..... |
| 120 | 14070 | 15150 | 16178 | 17228 | 18110 | 19181 | 20131 | 21040 | 22795 | 24487 | ..... | ..... |
| 110 | 12998 | 14006 | 14958 | 15924 | 16840 | 17718 | 18588 | 19418 | 21022 | ..... | ..... | ..... |
| 100 | 11945 | 12851 | 13726 | 14608 | 15440 | 16243 | 17022 | 17785 | ..... | ..... | ..... | ..... |
| 95 | 11462 | 12272 | 13105 | 13944 | 14736 | 15500 | 16252 | ..... | ..... | ..... | ..... | ..... |
| 90 | 10955 | 11725 | 12481 | 13277 | 14028 | 14754 | ..... | ..... | ..... | ..... | ..... | ..... |
| 85 | 10415 | 11142 | 11875 | 12606 | 13314 | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 80 | 9874 | 10565 | 11250 | 11932 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 75 | 9295 | 9943 | 10585 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 70 | 8684 | 9295 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 65 | 8062 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |

For $l_1$ and $l_2$ each $>142$ ft. $M = 1.5\, l_1 l_2 + 5700 \frac{l_2}{L}$



## TABLE 13

### MAXIMUM PIER REACTIONS BETWEEN EQUAL AND UNEQUAL SPANS, COOPER'S $E40$ LOADING

Values in Thousands of Pounds per Rail

SHORTER SEGMENT $l_1$

| Longer Segment $l_2$ | 0 | 5 | 10 | 15 | 20 | 25 | 30 | 35 | 40 | 45 | 50 | 55 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 250...... | 314 | 314 | 315 | 318 | 322 | 326 | 329 | 332 | 336 | 338 | 342 | 346 |
| 225...... | 287 | 287 | 290 | 294 | 298 | 301 | 304 | 306 | 309 | 312 | 317 | 321 |
| 200...... | 261 | 261 | 263 | 268 | 271 | 275 | 278 | 281 | 284 | 287 | 292 | 296 |
| 175...... | 234 | 234 | 236 | 241 | 244 | 248 | 251 | 254 | 258 | 262 | 266 | 269 |
| 160...... | 218 | 218 | 220 | 225 | 228 | 232 | 236 | 238 | 242 | 246 | 250 | 254 |
| 150...... | 207 | 207 | 210 | 214 | 218 | 222 | 225 | 229 | 231 | 234 | 239 | 244 |
| 140...... | 196 | 196 | 198 | 203 | 206 | 210 | 214 | 218 | 220 | 224 | 229 | 234 |
| 130...... | 185 | 185 | 187 | 192 | 196 | 201 | 203 | 208 | 210 | 214 | 219 | 224 |
| 120...... | 174 | 174 | 176 | 181 | 184 | 189 | 192 | 196 | 198 | 204 | 208 | 213 |
| 110...... | 162 | 162 | 165 | 170 | 173 | 178 | 181 | 185 | 188 | 193 | 198 | 202 |
| 100...... | 150 | 150 | 153 | 158 | 162 | 166 | 170 | 174 | 177 | 182 | 187 | 192 |
| 95...... | 144 | 144 | 146 | 151 | 155 | 160 | 163 | 168 | 173 | 178 | 182 | 188 |
| 90...... | 137 | 137 | 140 | 146 | 150 | 154 | 158 | 163 | 168 | 174 | 178 | 183 |
| 85...... | 131 | 131 | 134 | 139 | 142 | 148 | 152 | 158 | 163 | 168 | 174 | 178 |
| 80...... | 124 | 124 | 127 | 133 | 137 | 142 | 146 | 153 | 158 | 163 | 168 | 174 |
| 75...... | 118 | 118 | 122 | 126 | 130 | 135 | 140 | 146 | 152 | 158 | 162 | 167 |
| 70...... | 110 | 110 | 114 | 120 | 124 | 128 | 134 | 139 | 146 | 150 | 156 | 162 |
| 65...... | 104 | 104 | 107 | 112 | 118 | 122 | 126 | 133 | 139 | 144 | 149 | 155 |
| 60...... | 98 | 98 | 101 | 106 | 110 | 115 | 119 | 125 | 131 | 137 | 142 | 148 |
| 55...... | 93 | 93 | 95 | 99 | 103 | 108 | 113 | 118 | 125 | 130 | 134 | 141 |
| 50...... | 87 | 87 | 90 | 94 | 98 | 102 | 108 | 114 | 118 | 124 | 129 | ... |
| 45...... | 82 | 82 | 85 | 90 | 93 | 98 | 102 | 109 | 114 | 118 | ... | ... |
| 40...... | 75 | 75 | 79 | 84 | 88 | 92 | 98 | 102 | 108 | ... | ... | ... |
| 35...... | 69 | 69 | 74 | 78 | 82 | 87 | 92 | 98 | ... | ... | ... | ... |
| 30...... | 63 | 63 | 67 | 72 | 77 | 82 | 86 | ... | ... | ... | ... | ... |
| 25...... | 57 | 57 | 62 | 66 | 71 | 76 | ... | ... | ... | ... | ... | ... |
| 20...... | 50 | 50 | 56 | 60 | 66 | ... | ... | ... | ... | ... | ... | ... |
| 15...... | 40 | 40 | 50 | 55 | ... | ... | ... | ... | ... | ... | ... | ... |
| 10...... | 30 | 30 | 40 | ... | ... | ... | ... | ... | ... | ... | ... | ... |
| 5...... | 20 | 20 | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... |

For $l_1$ and $l_2$ each $>142$ ft. $R = L + \frac{3800}{l_1}$

TABLE 13.—*Continued*

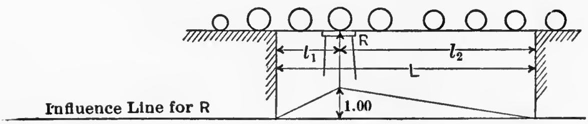
MAXIMUM PIER REACTIONS BETWEEN EQUAL AND UNEQUAL SPANS, COOPER'S $E40$ LOADING

Values in Thousands of Pounds per Rail

SHORTER SEGMENT $l_1$

| Longer Segment $l_2$ | 60 | 65 | 70 | 75 | 80 | 85 | 90 | 95 | 100 | 110 | 120 | 130 | 140 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 250 | 350 | 356 | 359 | 365 | 370 | 374 | 379 | 382 | 387 | 395 | 402 | 410 | 417 |
| 225 | 326 | 330 | 334 | 340 | 345 | 350 | 354 | 358 | 362 | 370 | 377 | 385 | 392 |
| 200 | 300 | 305 | 309 | 314 | 320 | 324 | 329 | 333 | 337 | 345 | 352 | 359 | 367 |
| 175 | 274 | 279 | 284 | 290 | 294 | 300 | 303 | 308 | 312 | 319 | 327 | 334 | 342 |
| 160 | 258 | 264 | 269 | 274 | 280 | 284 | 289 | 293 | 297 | 305 | 312 | 320 | 328 |
| 150 | 248 | 254 | 259 | 264 | 269 | 274 | 278 | 282 | 287 | 295 | 302 | 310 | 318 |
| 140 | 238 | 242 | 249 | 253 | 259 | 264 | 270 | 273 | 277 | 284 | 292 | 299 | 308 |
| 130 | 229 | 233 | 239 | 243 | 250 | 254 | 258 | 262 | 267 | 274 | 282 | 290 | ... |
| 120 | 218 | 222 | 228 | 233 | 239 | 242 | 248 | 253 | 257 | 265 | 272 | ... | ... |
| 110 | 207 | 212 | 218 | 223 | 230 | 234 | 238 | 243 | 247 | 255 | ... | ... | ... |
| 100 | 197 | 202 | 208 | 214 | 219 | 224 | 229 | 233 | 238 | ... | ... | ... | ... |
| 95 | 192 | 198 | 203 | 208 | 214 | 219 | 223 | 229 | ... | ... | ... | ... | ... |
| 90 | 188 | 194 | 198 | 203 | 209 | 214 | 218 | ... | ... | ... | ... | ... | ... |
| 85 | 183 | 189 | 194 | 198 | 204 | 209 | ... | ... | ... | ... | ... | ... | ... |
| 80 | 178 | 184 | 188 | 194 | 199 | ... | ... | ... | ... | ... | ... | ... | ... |
| 75 | 173 | 178 | 183 | 188 | ... | ... | ... | ... | ... | ... | ... | ... | ... |
| 70 | 166 | 171 | 178 | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... |
| 65 | 160 | 165 | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... |
| 60 | 153 | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... |

For $l_1$ and $l_2$ each $>142$ ft. $R = L + \dfrac{3800}{l_1}$



## TABLE 14

### Maximum Pier Reactions Between Equal and Unequal Spans, Cooper's $E50$ Loading

Values in Thousands of Pounds per Rail

| Longer Segment $l_2$ | Shorter Segment $l_1$: 0 | 5 | 10 | 15 | 20 | 25 | 30 | 35 | 40 | 45 | 50 | 55 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 250...... | 392 | 392 | 394 | 398 | 403 | 407 | 411 | 415 | 420 | 423 | 428 | 432 |
| 225...... | 359 | 359 | 362 | 367 | 372 | 376 | 380 | 383 | 386 | 390 | 396 | 401 |
| 200...... | 326 | 326 | 329 | 335 | 339 | 344 | 347 | 351 | 355 | 359 | 365 | 370 |
| 175...... | 293 | 293 | 295 | 301 | 305 | 310 | 314 | 318 | 323 | 327 | 332 | 336 |
| 160...... | 273 | 273 | 275 | 281 | 285 | 290 | 295 | 298 | 302 | 307 | 313 | 318 |
| 150...... | 259 | 259 | 262 | 267 | 272 | 277 | 281 | 286 | 289 | 293 | 299 | 305 |
| 140...... | 245 | 245 | 248 | 254 | 258 | 263 | 268 | 273 | 275 | 280 | 286 | 293 |
| 130...... | 231 | 231 | 234 | 240 | 245 | 251 | 254 | 260 | 262 | 268 | 274 | 280 |
| 120...... | 217 | 217 | 220 | 226 | 230 | 236 | 240 | 245 | 248 | 255 | 260 | 266 |
| 110...... | 202 | 202 | 206 | 212 | 216 | 222 | 226 | 231 | 235 | 241 | 247 | 253 |
| 100...... | 187 | 187 | 191 | 197 | 202 | 208 | 212 | 218 | 221 | 227 | 234 | 240 |
| 95...... | 180 | 180 | 183 | 189 | 194 | 200 | 204 | 210 | 216 | 222 | 228 | 235 |
| 90...... | 171 | 171 | 175 | 182 | 187 | 192 | 197 | 204 | 210 | 218 | 223 | 229 |
| 85...... | 164 | 164 | 168 | 174 | 178 | 185 | 190 | 198 | 204 | 210 | 217 | 223 |
| 80...... | 155 | 155 | 159 | 166 | 171 | 177 | 183 | 191 | 197 | 204 | 210 | 217 |
| 75...... | 147 | 147 | 152 | 158 | 163 | 169 | 175 | 183 | 190 | 197 | 203 | 209 |
| 70...... | 138 | 138 | 143 | 150 | 155 | 160 | 167 | 174 | 182 | 188 | 195 | 202 |
| 65...... | 130 | 130 | 134 | 140 | 147 | 152 | 158 | 166 | 174 | 180 | 186 | 194 |
| 60...... | 123 | 123 | 126 | 132 | 137 | 144 | 149 | 156 | 164 | 171 | 178 | 185 |
| 55...... | 116 | 116 | 119 | 124 | 129 | 135 | 141 | 148 | 156 | 162 | 168 | 176 |
| 50...... | 109 | 109 | 112 | 118 | 122 | 128 | 135 | 142 | 148 | 155 | 161 | ... |
| 45...... | 102 | 102 | 106 | 112 | 116 | 122 | 128 | 136 | 142 | 148 | ... | ... |
| 40...... | 94 | 94 | 99 | 105 | 110 | 115 | 122 | 128 | 135 | ... | ... | ... |
| 35...... | 86 | 86 | 92 | 98 | 103 | 109 | 115 | 122 | ... | ... | ... | ... |
| 30...... | 79 | 79 | 84 | 90 | 96 | 102 | 108 | ... | ... | ... | ... | ... |
| 25...... | 71 | 71 | 77 | 83 | 89 | 95 | ... | ... | ... | ... | ... | ... |
| 20...... | 63 | 63 | 70 | 75 | 82 | ... | ... | ... | ... | ... | ... | ... |
| 15...... | 50 | 50 | 62 | 69 | ... | ... | ... | ... | ... | ... | ... | ... |
| 10...... | 38 | 38 | 50 | ... | ... | ... | ... | ... | ... | ... | ... | ... |
| 5...... | 25 | 25 | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... |

For $l_1$ and $l_2$ each $>142$ ft. $R = 1.25\,L + \frac{4750}{l_1}$

TABLE 14.—*Continued*

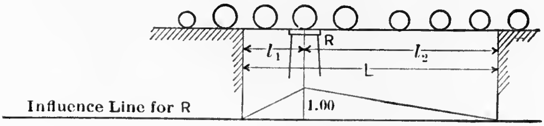
MAXIMUM PIER REACTIONS BETWEEN EQUAL AND UNEQUAL SPANS, COOPER'S *E*50 LOADING

Values in Thousands of Pounds per Rail

SHORTER SEGMENT $l_1$

| | 60 | 65 | 70 | 75 | 80 | 85 | 90 | 95 | 100 | 110 | 120 | 130 | 140 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 250... | 437 | 445 | 449 | 456 | 463 | 468 | 474 | 478 | 484 | 494 | 502 | 512 | 521 |
| 225... | 407 | 413 | 418 | 425 | 431 | 437 | 442 | 448 | 452 | 462 | 471 | 481 | 490 |
| 200... | 375 | 381 | 386 | 393 | 400 | 405 | 411 | 416 | 421 | 431 | 440 | 449 | 459 |
| 175... | 343 | 349 | 355 | 362 | 368 | 375 | 379 | 385 | 390 | 399 | 409 | 418 | 427 |
| 160... | 323 | 330 | 336 | 343 | 350 | 355 | 361 | 366 | 371 | 381 | 390 | 400 | 410 |
| 150... | 310 | 317 | 324 | 330 | 336 | 343 | 348 | 353 | 359 | 369 | 378 | 387 | 397 |
| 140... | 298 | 303 | 311 | 316 | 324 | 330 | 337 | 341 | 346 | 355 | 365 | 374 | 385 |
| 130... | 286 | 291 | 299 | 304 | 312 | 317 | 323 | 328 | 334 | 343 | 352 | 362 | ... |
| 120... | 272 | 278 | 285 | 291 | 299 | 303 | 310 | 316 | 321 | 331 | 340 | ... | ... |
| 110... | 259 | 265 | 273 | 279 | 287 | 292 | 298 | 304 | 309 | 319 | ... | ... | ... |
| 100... | 246 | 253 | 260 | 267 | 274 | 280 | 286 | 291 | 296 | ... | ... | ... | ... |
| 95... | 240 | 247 | 254 | 260 | 267 | 274 | 279 | 286 | ... | ... | ... | ... | ... |
| 90... | 235 | 242 | 248 | 254 | 261 | 268 | 273 | ... | ... | ... | ... | ... | ... |
| 85... | 229 | 236 | 242 | 248 | 255 | 261 | ... | ... | ... | ... | ... | ... | ... |
| 80... | 223 | 230 | 235 | 242 | 249 | ... | ... | ... | ... | ... | ... | ... | ... |
| 75... | 216 | 222 | 229 | 235 | ... | ... | ... | ... | ... | ... | ... | ... | ... |
| 70... | 208 | 214 | 222 | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... |
| 65... | 200 | 206 | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... |
| 60... | 191 | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... |

For $l_1$ and $l_2$ each $>142$ ft. $R = 1.25\,L + \dfrac{4750}{l_1}$



## TABLE 15

### Maximum Pier Reactions Between Equal and Unequal Spans, Cooper's $E60$ Loading

Values in Thousands of Pounds per Rail

| Longer Segment $l_2$ | Shorter Segment $l_1$: 0 | 5 | 10 | 15 | 20 | 25 | 30 | 35 | 40 | 45 | 50 | 55 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 250 | 470 | 470 | 473 | 478 | 484 | 488 | 493 | 498 | 504 | 508 | 514 | 518 |
| 225 | 431 | 431 | 434 | 440 | 446 | 451 | 456 | 460 | 463 | 468 | 475 | 481 |
| 200 | 391 | 391 | 395 | 402 | 407 | 413 | 417 | 421 | 426 | 431 | 438 | 444 |
| 175 | 352 | 352 | 354 | 361 | 366 | 372 | 377 | 382 | 388 | 392 | 398 | 403 |
| 160 | 328 | 328 | 330 | 337 | 342 | 348 | 354 | 358 | 362 | 368 | 376 | 382 |
| 150 | 311 | 311 | 314 | 320 | 326 | 332 | 337 | 343 | 347 | 352 | 359 | 366 |
| 140 | 294 | 294 | 298 | 305 | 310 | 316 | 322 | 328 | 330 | 336 | 343 | 352 |
| 130 | 277 | 277 | 281 | 288 | 294 | 301 | 305 | 312 | 314 | 322 | 329 | 336 |
| 120 | 260 | 260 | 264 | 271 | 276 | 283 | 288 | 294 | 298 | 306 | 312 | 319 |
| 110 | 242 | 242 | 247 | 254 | 259 | 266 | 271 | 277 | 282 | 289 | 296 | 304 |
| 100 | 224 | 224 | 229 | 236 | 242 | 250 | 254 | 262 | 265 | 272 | 281 | 288 |
| 95 | 216 | 216 | 220 | 227 | 233 | 240 | 245 | 252 | 259 | 266 | 274 | 282 |
| 90 | 205 | 205 | 210 | 218 | 224 | 230 | 236 | 245 | 252 | 262 | 268 | 275 |
| 85 | 197 | 197 | 202 | 209 | 214 | 222 | 228 | 238 | 245 | 252 | 260 | 268 |
| 80 | 186 | 186 | 191 | 199 | 205 | 212 | 220 | 229 | 236 | 245 | 252 | 260 |
| 75 | 176 | 176 | 182 | 190 | 196 | 203 | 210 | 220 | 228 | 236 | 244 | 251 |
| 70 | 166 | 166 | 172 | 180 | 186 | 192 | 200 | 209 | 218 | 226 | 234 | 242 |
| 65 | 156 | 156 | 161 | 168 | 176 | 182 | 190 | 199 | 209 | 216 | 223 | 233 |
| 60 | 148 | 148 | 151 | 158 | 164 | 173 | 179 | 187 | 197 | 205 | 214 | 222 |
| 55 | 139 | 139 | 143 | 149 | 155 | 162 | 169 | 178 | 187 | 194 | 202 | 211 |
| 50 | 131 | 131 | 134 | 142 | 146 | 154 | 162 | 170 | 178 | 186 | 193 | ... |
| 45 | 122 | 122 | 127 | 134 | 139 | 146 | 154 | 163 | 170 | 178 | ... | ... |
| 40 | 113 | 113 | 119 | 126 | 132 | 138 | 146 | 154 | 162 | ... | ... | ... |
| 35 | 103 | 103 | 110 | 118 | 124 | 131 | 138 | 146 | ... | ... | ... | ... |
| 30 | 95 | 95 | 101 | 108 | 115 | 122 | 130 | ... | ... | ... | ... | ... |
| 25 | 85 | 85 | 92 | 100 | 107 | 114 | ... | ... | ... | ... | ... | ... |
| 20 | 76 | 76 | 84 | 90 | 98 | ... | ... | ... | ... | ... | ... | ... |
| 15 | 60 | 60 | 74 | 83 | ... | ... | ... | ... | ... | ... | ... | ... |
| 10 | 46 | 46 | 60 | ... | ... | ... | ... | ... | ... | ... | ... | ... |
| 5 | 30 | 30 | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... |

For $l_1$ and $l_2$ each $>142$ ft. $R = 1.5\,L + \frac{5700}{l_1}$

## TABLE 15.—*Continued*

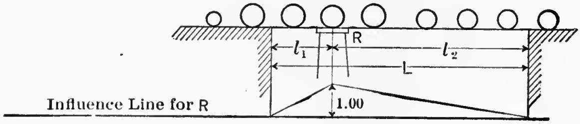
MAXIMUM PIER REACTIONS BETWEEN EQUAL AND UNEQUAL SPANS, COOPER'S *E*60 LOADING

Values in Thousands of Pounds per Rail

SHORTER SEGMENT $l_1$

| | 60 | 65 | 70 | 75 | 80 | 85 | 90 | 95 | 100 | 110 | 120 | 130 | 140 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 250... | 524 | 534 | 539 | 547 | 556 | 562 | 569 | 574 | 581 | 593 | 602 | 614 | 625 |
| 225... | 488 | 496 | 502 | 510 | 517 | 524 | 530 | 538 | 542 | 554 | 565 | 577 | 588 |
| 200... | 450 | 457 | 463 | 472 | 480 | 486 | 493 | 499 | 505 | 517 | 528 | 539 | 551 |
| 175... | 412 | 419 | 426 | 434 | 442 | 450 | 455 | 462 | 468 | 479 | 491 | 502 | 512 |
| 160... | 388 | 396 | 403 | 412 | 420 | 426 | 433 | 439 | 445 | 457 | 468 | 480 | 492 |
| 150... | 372 | 380 | 389 | 396 | 403 | 412 | 418 | 424 | 431 | 443 | 454 | 464 | 476 |
| 140... | 358 | 364 | 373 | 379 | 389 | 396 | 404 | 409 | 415 | 426 | 438 | 449 | 462 |
| 130... | 343 | 349 | 359 | 365 | 374 | 380 | 388 | 394 | 401 | 412 | 422 | 434 | ... |
| 120... | 326 | 334 | 342 | 349 | 359 | 364 | 372 | 379 | 385 | 397 | 408 | ... | ... |
| 110... | 311 | 318 | 328 | 335 | 344 | 350 | 358 | 365 | 371 | 383 | ... | ... | ... |
| 100... | 295 | 304 | 312 | 320 | 329 | 336 | 343 | 349 | 356 | ... | ... | ... | ... |
| 95... | 288 | 296 | 305 | 312 | 320 | 329 | 335 | 343 | ... | ... | ... | ... | ... |
| 90... | 282 | 290 | 298 | 305 | 313 | 322 | 328 | ... | ... | ... | ... | ... | ... |
| 85... | 275 | 283 | 290 | 298 | 306 | 313 | ... | ... | ... | ... | ... | ... | ... |
| 80... | 268 | 276 | 282 | 290 | 299 | ... | ... | ... | ... | ... | ... | ... | ... |
| 75... | 259 | 266 | 275 | 282 | ... | ... | ... | ... | ... | ... | ... | ... | ... |
| 70... | 250 | 257 | 266 | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... |
| 65... | 240 | 247 | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... |
| 60.. | 229 | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... |

For $l_1$ and $l_2$ each $>142$ ft. $R = 1.5\,L + \dfrac{5700}{l_1}$



## TABLE 16

### EQUIVALENT UNIFORM LOADS FOR COOPER'S $E$40 LOADING

Values in Pounds per Lineal Foot per Rail

| Longer Segment $l_2$ | Shorter Segment $l_1$: 0 | 5 | 10 | 15 | 20 | 25 | 30 | 35 | 40 | 45 | 50 | 55 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 250...... | 2500 | 2450 | 2430 | 2410 | 2380 | 2370 | 2350 | 2330 | 2310 | 2300 | 2290 | 2270 |
| 225...... | 2550 | 2500 | 2460 | 2450 | 2430 | 2400 | 2380 | 2360 | 2340 | 2320 | 2310 | 2300 |
| 200...... | 2610 | 2540 | 2500 | 2490 | 2460 | 2440 | 2420 | 2390 | 2370 | 2350 | 2340 | 2320 |
| 175...... | 2680 | 2610 | 2550 | 2540 | 2510 | 2490 | 2460 | 2420 | 2400 | 2380 | 2360 | 2340 |
| 160...... | 2730 | 2630 | 2590 | 2570 | 2540 | 2510 | 2480 | 2450 | 2420 | 2400 | 2380 | 2370 |
| 150...... | 2760 | 2670 | 2620 | 2590 | 2570 | 2540 | 2500 | 2460 | 2430 | 2420 | 2400 | 2380 |
| 140...... | 2800 | 2700 | 2650 | 2620 | 2580 | 2560 | 2520 | 2490 | 2450 | 2430 | 2420 | 2400 |
| 130...... | 2850 | 2740 | 2670 | 2650 | 2610 | 2580 | 2540 | 2510 | 2470 | 2450 | 2430 | 2420 |
| 120...... | 2900 | 2770 | 2710 | 2680 | 2640 | 2610 | 2560 | 2530 | 2490 | 2460 | 2450 | 2430 |
| 110...... | 2940 | 2810 | 2740 | 2710 | 2660 | 2630 | 2580 | 2550 | 2500 | 2490 | 2460 | 2460 |
| 100...... | 3000 | 2850 | 2780 | 2740 | 2690 | 2660 | 2610 | 2570 | 2530 | 2510 | 2500 | 2480 |
| 95...... | 3020 | 2880 | 2800 | 2760 | 2700 | 2670 | 2620 | 2580 | 2560 | 2540 | 2520 | 2500 |
| 90...... | 3050 | 2890 | 2810 | 2770 | 2720 | 2680 | 2630 | 2620 | 2590 | 2570 | 2550 | 2540 |
| 85...... | 3080 | 2920 | 2820 | 2780 | 2730 | 2700 | 2640 | 2640 | 2620 | 2580 | 2570 | 2550 |
| 80...... | 3110 | 2920 | 2840 | 2790 | 2740 | 2710 | 2670 | 2660 | 2620 | 2610 | 2580 | 2570 |
| 75...... | 3140 | 2940 | 2860 | 2800 | 2740 | 2700 | 2670 | 2660 | 2640 | 2620 | 2600 | 2580 |
| 70...... | 3160 | 2940 | 2870 | 2810 | 2750 | 2700 | 2670 | 2660 | 2650 | 2620 | 2600 | 2580 |
| 65...... | 3190 | 2960 | 2870 | 2810 | 2760 | 2700 | 2670 | 2660 | 2650 | 2620 | 2600 | 2580 |
| 60...... | 3270 | 3020 | 2880 | 2820 | 2750 | 2700 | 2660 | 2640 | 2630 | 2610 | 2590 | 2580 |
| 55...... | 3370 | 3090 | 2930 | 2840 | 2760 | 2700 | 2660 | 2650 | 2620 | 2600 | 2560 | 2550 |
| 50...... | 3490 | 3180 | 3000 | 2910 | 2800 | 2740 | 2700 | 2670 | 2630 | 2600 | 2580 | .... |
| 45...... | 3630 | 3260 | 3080 | 2980 | 2870 | 2780 | 2740 | 2710 | 2670 | 2640 | .... | .... |
| 40...... | 3770 | 3350 | 3180 | 3060 | 2930 | 2840 | 2780 | 2740 | 2700 | .... | .... | .... |
| 35...... | 3960 | 3450 | 3260 | 3120 | 3010 | 2900 | 2840 | 2790 | .... | .... | .... | .... |
| 30...... | 4200 | 3610 | 3380 | 3200 | 3060 | 2960 | 2880 | .... | .... | .... | .... | .... |
| 25...... | 4540 | 3770 | 3520 | 3320 | 3150 | 3020 | .... | .... | .... | .... | .... | .... |
| 20...... | 5000 | 4000 | 3730 | 3450 | 3280 | .... | .... | .... | .... | .... | .... | .... |
| 15...... | 5336 | 4000 | 4000 | 3650 | .... | .... | .... | .... | .... | .... | .... | .... |
| 10...... | 6000 | 4000 | 4000 | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| 5...... | 8000 | 4000 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |

For $l_1$ and $l_2$ each $>142$ ft. $q = \left(2.0 + \frac{7600}{l_1 L}\right) 1000$

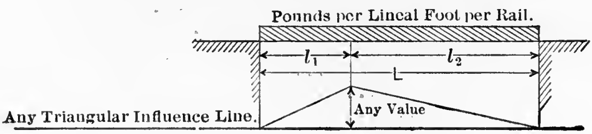
## TABLE 16.—*Continued*

### Equivalent Uniform Loads for Cooper's *E*40 Loading

Values in Pounds per Lineal Foot per Rail

| | Shorter Segment $l_1$ | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 60 | 65 | 70 | 75 | 80 | 85 | 90 | 95 | 100 | 110 | 120 | 130 | 140 |
| 250...... | 2260 | 2260 | 2250 | 2250 | 2240 | 2230 | 2220 | 2220 | 2210 | 2200 | 2180 | 2160 | 2140 |
| 225...... | 2290 | 2280 | 2270 | 2270 | 2260 | 2260 | 2250 | 2240 | 2220 | 2220 | 2180 | 2170 | 2150 |
| 200...... | 2310 | 2300 | 2290 | 2290 | 2280 | 2280 | 2270 | 2260 | 2250 | 2230 | 2200 | 2180 | 2160 |
| 175...... | 2340 | 2320 | 2320 | 2320 | 2310 | 2300 | 2290 | 2280 | 2270 | 2240 | 2210 | 2200 | 2180 |
| 160...... | 2350 | 2340 | 2340 | 2340 | 2330 | 2320 | 2310 | 2300 | 2280 | 2260 | 2230 | 2210 | 2180 |
| 150...... | 2370 | 2350 | 2360 | 2350 | 2340 | 2340 | 2330 | 2300 | 2300 | 2270 | 2240 | 2220 | 2190 |
| 140...... | 2380 | 2380 | 2370 | 2360 | 2360 | 2350 | 2340 | 2320 | 2310 | 2280 | 2250 | 2220 | 2200 |
| 130...... | 2400 | 2390 | 2390 | 2380 | 2380 | 2370 | 2350 | 2340 | 2330 | 2290 | 2260 | 2230 | .... |
| 120...... | 2420 | 2410 | 2410 | 2400 | 2400 | 2370 | 2370 | 2350 | 2340 | 2300 | 2270 | .... | .... |
| 110...... | 2440 | 2420 | 2420 | 2420 | 2420 | 2400 | 2390 | 2380 | 2350 | 2320 | .... | .... | .... |
| 100...... | 2460 | 2460 | 2450 | 2440 | 2440 | 2420 | 2410 | 2390 | 2380 | .... | .... | .... | .... |
| 95...... | 2500 | 2480 | 2460 | 2460 | 2450 | 2440 | 2420 | 2400 | .... | .... | .... | .... | .... |
| 90...... | 2510 | 2500 | 2480 | 2460 | 2460 | 2450 | 2430 | .... | .... | .... | .... | .... | .... |
| 85...... | 2530 | 2510 | 2500 | 2490 | 2470 | 2460 | .... | .... | .... | .... | .... | .... | .... |
| 80...... | 2550 | 2540 | 2520 | 2500 | 2490 | .... | .... | .... | .... | .... | .... | .... | .... |
| 75...... | 2560 | 2540 | 2530 | 2510 | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| 70...... | 2560 | 2540 | 2530 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| 65...... | 2560 | 2540 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| 60...... | 2550 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |

For $l_1$ and $l_2$ each $>142$ ft. $q = \left(2.0 + \frac{7600}{l_1 L}\right) 1000$



## TABLE 17

### Equivalent Uniform Loads for Cooper's $E$50 Loading

Values in Pounds per Lineal Foot per Rail

| Longer Segment $l_2$ | Shorter Segment $l_1$: 0 | 5 | 10 | 15 | 20 | 25 | 30 | 35 | 40 | 45 | 50 | 55 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 250...... | 3130 | 3060 | 3040 | 3010 | 2980 | 2960 | 2940 | 2910 | 2890 | 2870 | 2860 | 2840 |
| 225...... | 3190 | 3120 | 3080 | 3060 | 3040 | 3000 | 2980 | 2950 | 2920 | 2900 | 2890 | 2870 |
| 200...... | 3265 | 3180 | 3130 | 3110 | 3080 | 3050 | 3020 | 2990 | 2960 | 2940 | 2920 | 2900 |
| 175...... | 3350 | 3260 | 3190 | 3170 | 3140 | 3110 | 3070 | 3030 | 3000 | 2970 | 2950 | 2930 |
| 160...... | 3410 | 3290 | 3240 | 3210 | 3170 | 3140 | 3100 | 3060 | 3020 | 3000 | 2980 | 2960 |
| 150...... | 3455 | 3340 | 3270 | 3240 | 3210 | 3170 | 3130 | 3080 | 3040 | 3020 | 3000 | 2980 |
| 140...... | 3505 | 3380 | 3305 | 3275 | 3230 | 3195 | 3150 | 3110 | 3064 | 3040 | 3018 | 3000 |
| 130...... | 3560 | 3420 | 3340 | 3310 | 3260 | 3225 | 3175 | 3135 | 3085 | 3060 | 3039 | 3020 |
| 120...... | 3620 | 3460 | 3385 | 3350 | 3295 | 3255 | 3200 | 3160 | 3106 | 3080 | 3060 | 3040 |
| 110...... | 3680 | 3510 | 3430 | 3385 | 3330 | 3285 | 3225 | 3185 | 3133 | 3105 | 3083 | 3065 |
| 100...... | 3750 | 3560 | 3470 | 3425 | 3360 | 3320 | 3260 | 3210 | 3158 | 3140 | 3117 | 3095 |
| 95...... | 3780 | 3600 | 3500 | 3445 | 3375 | 3340 | 3275 | 3225 | 3200 | 3175 | 3153 | 3130 |
| 90...... | 3810 | 3610 | 3510 | 3455 | 3395 | 3350 | 3290 | 3265 | 3237 | 3210 | 3186 | 3165 |
| 85...... | 3850 | 3650 | 3530 | 3470 | 3405 | 3370 | 3300 | 3295 | 3266 | 3225 | 3210 | 3185 |
| 80...... | 3885 | 3650 | 3545 | 3480 | 3415 | 3385 | 3335 | 3315 | 3284 | 3255 | 3232 | 3210 |
| 75...... | 3920 | 3670 | 3565 | 3495 | 3425 | 3380 | 3340 | 3325 | 3303 | 3275 | 3250 | 3225 |
| 70...... | 3945 | 3680 | 3585 | 3510 | 3435 | 3380 | 3340 | 3320 | 3308 | 3280 | 3252 | 3225 |
| 65...... | 3990 | 3700 | 3580 | 3505 | 3445 | 3375 | 3335 | 3325 | 3305 | 3270 | 3246 | 3220 |
| 60...... | 4085 | 3780 | 3595 | 3515 | 3435 | 3375 | 3315 | 3300 | 3286 | 3260 | 3237 | 3215 |
| 55...... | 4215 | 3860 | 3660 | 3550 | 3450 | 3380 | 3325 | 3305 | 3277 | 3245 | 3194 | 3190 |
| 50...... | 4360 | 3970 | 3750 | 3635 | 3495 | 3425 | 3370 | 3335 | 3293 | 3250 | 3219 | .... |
| 45...... | 4540 | 4080 | 3850 | 3720 | 3585 | 3480 | 3420 | 3390 | 3339 | 3295 | .... | .... |
| 40...... | 4715 | 4190 | 3975 | 3825 | 3660 | 3550 | 3475 | 3430 | 3375 | .... | .... | .... |
| 35...... | 4945 | 4310 | 4080 | 3900 | 3760 | 3630 | 3545 | 3485 | .... | .... | .... | .... |
| 30...... | 5255 | 4510 | 4215 | 4000 | 3825 | 3695 | 3595 | .... | .... | .... | .... | .... |
| 25...... | 5680 | 4710 | 4400 | 4150 | 3935 | 3780 | .... | .... | .... | .... | .... | .... |
| 20...... | 6250 | 5000 | 4660 | 4315 | 4100 | .... | .... | .... | .... | .... | .... | .... |
| 15...... | 6670 | 5000 | 5000 | 4560 | .... | .... | .... | .... | .... | .... | .... | .... |
| 10...... | 7500 | 5000 | 5000 | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| 5...... | 10000 | 5000 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |

For $l_1$ and $l_2$ each $>142$ ft. $q = \left(2.5 + \frac{9500}{l_1 L}\right) 1000$

TABLE 17.—*Continued*

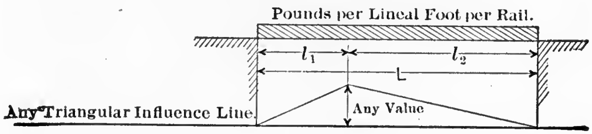
EQUIVALENT UNIFORM LOADS FOR COOPER'S *E*50 LOADING

Values in Pounds per Lineal Foot per Rail

SHORTER SEGMENT $l_1$

| | 60 | 65 | 70 | 75 | 80 | 85 | 90 | 95 | 100 | 110 | 120 | 130 | 140 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 250...... | 2830 | 2820 | 2810 | 2810 | 2800 | 2790 | 2780 | 2770 | 2760 | 2750 | 2720 | 2700 | 2680 |
| 225...... | 2860 | 2850 | 2840 | 2840 | 2830 | 2820 | 2810 | 2800 | 2780 | 2770 | 2730 | 2710 | 2690 |
| 200...... | 2890 | 2870 | 2860 | 2860 | 2850 | 2850 | 2840 | 2820 | 2810 | 2790 | 2750 | 2720 | 2700 |
| 175...... | 2920 | 2900 | 2900 | 2900 | 2890 | 2880 | 2860 | 2850 | 2840 | 2800 | 2760 | 2750 | 2720 |
| 160...... | 2940 | 2930 | 2920 | 2920 | 2910 | 2900 | 2890 | 2870 | 2850 | 2820 | 2790 | 2760 | 2730 |
| 150...... | 2960 | 2940 | 2950 | 2940 | 2930 | 2920 | 2910 | 2880 | 2870 | 2840 | 2800 | 2770 | 2740 |
| 140...... | 2980 | 2965 | 2960 | 2950 | 2950 | 2940 | 2920 | 2900 | 2890 | 2850 | 2810 | 2775 | 2750 |
| 130...... | 3000 | 2985 | 2985 | 2975 | 2970 | 2955 | 2940 | 2920 | 2905 | 2860 | 2820 | 2785 | .... |
| 120...... | 3020 | 3005 | 3005 | 2995 | 2995 | 2960 | 2960 | 2940 | 2920 | 2880 | 2835 | .... | .... |
| 110...... | 3045 | 3030 | 3030 | 3020 | 3015 | 3000 | 2985 | 2965 | 2940 | 2895 | .... | .... | .... |
| 100...... | 3080 | 3065 | 3060 | 3050 | 3045 | 3030 | 3010 | 2985 | 2965 | .... | .... | .... | .... |
| 95...... | 3115 | 3095 | 3075 | 3065 | 3060 | 3050 | 3020 | 3001 | .... | .... | .... | .... | .... |
| 90...... | 3140 | 3120 | 3100 | 3080 | 3075 | 3060 | 3035 | .... | .... | .... | .... | .... | .... |
| 85...... | 3160 | 3140 | 3120 | 3105 | 3090 | 3070 | .... | .... | .... | .... | .... | .... | .... |
| 80...... | 3185 | 3165 | 3145 | 3125 | 3110 | .... | .... | .... | .... | .... | .... | .... | .... |
| 75...... | 3200 | 3180 | 3155 | 3140 | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| 70...... | 3200 | 3180 | 3160 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| 65...... | 3200 | 3180 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| 60...... | 3190 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |

For $l_1$ and $l_2$ each $>142$ ft. $q = \left(2.5 + \frac{9500}{l_1 L}\right) 1000$



## TABLE 18

### Equivalent Uniform Loads for Cooper's $E$60 Loading

Values in Pounds per Lineal Foot per Rail

Shorter Segment $l_1$

| Longer Segment $l_2$ | 0 | 5 | 10 | 15 | 20 | 25 | 30 | 35 | 40 | 45 | 50 | 55 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 250 | 3760 | 3670 | 3650 | 3610 | 3580 | 3550 | 3530 | 3490 | 3470 | 3440 | 3430 | 3410 |
| 225 | 3830 | 3740 | 3700 | 3670 | 3650 | 3600 | 3580 | 3540 | 3500 | 3480 | 3470 | 3440 |
| 200 | 3920 | 3820 | 3760 | 3730 | 3700 | 3660 | 3620 | 3590 | 3550 | 3530 | 3500 | 3480 |
| 175 | 4020 | 3910 | 3830 | 3800 | 3770 | 3730 | 3680 | 3640 | 3600 | 3560 | 3540 | 3520 |
| 160 | 4090 | 3950 | 3890 | 3850 | 3800 | 3770 | 3720 | 3670 | 3620 | 3600 | 3580 | 3550 |
| 150 | 4150 | 4010 | 3920 | 3890 | 3850 | 3800 | 3760 | 3700 | 3650 | 3620 | 3600 | 3580 |
| 140 | 4210 | 4060 | 3970 | 3940 | 3880 | 3840 | 3780 | 3730 | 3680 | 3650 | 3630 | 3600 |
| 130 | 4270 | 4110 | 4010 | 3970 | 3910 | 3850 | 3820 | 3770 | 3710 | 3670 | 3650 | 3620 |
| 120 | 4340 | 4150 | 4070 | 4020 | 3960 | 3910 | 3840 | 3790 | 3730 | 3700 | 3670 | 3650 |
| 110 | 4420 | 4210 | 4120 | 4070 | 4000 | 3950 | 3880 | 3830 | 3760 | 3760 | 3700 | 3680 |
| 100 | 4500 | 4270 | 4160 | 4120 | 4030 | 3980 | 3910 | 3850 | 3790 | 3770 | 3740 | 3720 |
| 95 | 4540 | 4320 | 4200 | 4140 | 4060 | 4010 | 3940 | 3880 | 3840 | 3820 | 3780 | 3760 |
| 90 | 4570 | 4330 | 4210 | 4150 | 4080 | 4020 | 3950 | 3920 | 3890 | 3850 | 3830 | 3800 |
| 85 | 4620 | 4380 | 4240 | 4160 | 4080 | 4040 | 3960 | 3960 | 3920 | 3880 | 3850 | 3830 |
| 80 | 4660 | 4380 | 4260 | 4180 | 4100 | 4070 | 4010 | 3980 | 3940 | 3910 | 3880 | 3850 |
| 75 | 4700 | 4400 | 4280 | 4200 | 4120 | 4060 | 4010 | 4000 | 3960 | 3940 | 3900 | 3870 |
| 70 | 4730 | 4420 | 4310 | 4210 | 4130 | 4060 | 4010 | 3980 | 3970 | 3940 | 3900 | 3870 |
| 65 | 4790 | 4440 | 4300 | 4210 | 4140 | 4060 | 4010 | 4000 | 3970 | 3920 | 3900 | 3860 |
| 60 | 4900 | 4540 | 4320 | 4220 | 4130 | 4060 | 3980 | 3960 | 3950 | 3910 | 3890 | 3860 |
| 55 | 5060 | 4630 | 4390 | 4260 | 4140 | 4060 | 4000 | 3970 | 3940 | 3900 | 3840 | 3830 |
| 50 | 5230 | 4760 | 4500 | 4370 | 4200 | 4120 | 4040 | 4010 | 3950 | 3900 | 3860 | .... |
| 45 | 5450 | 4900 | 4620 | 4460 | 4310 | 4180 | 4100 | 4070 | 4010 | 3960 | .... | .... |
| 40 | 5660 | 5030 | 4780 | 4600 | 4390 | 4260 | 4180 | 4120 | 4060 | .... | .... | .... |
| 35 | 5930 | 5170 | 4900 | 4680 | 4510 | 4360 | 4260 | 4190 | .... | .... | .... | .... |
| 30 | 6310 | 5410 | 5060 | 4800 | 4600 | 4440 | 4320 | .... | .... | .... | .... | .... |
| 25 | 6820 | 5650 | 5280 | 4980 | 4730 | 4540 | .... | .... | .... | .... | .... | .... |
| 20 | 7500 | 6000 | 5590 | 5180 | 4920 | .... | .... | .... | .... | .... | .... | .... |
| 15 | 8000 | 6000 | 6000 | 5470 | .... | .... | .... | .... | .... | .... | .... | .... |
| 10 | 9000 | 6000 | 6000 | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| 5 | 12000 | 6000 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |

For $l_1$ and $l_2$ each $>142$ ft. $q = \left(3.0 + \frac{11400}{l_1 L}\right) 1000$

## TABLE 18.—*Continued*

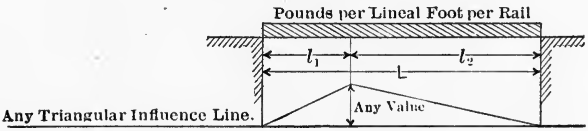
### Equivalent Uniform Loads for Cooper's *E*60 Loading

Values in Pounds per Lineal Foot per Rail

Shorter Segment $l_1$

| | 60 | 65 | 70 | 75 | 80 | 85 | 90 | 95 | 100 | 110 | 120 | 130 | 140 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 250...... | 3400 | 3380 | 3370 | 3370 | 3360 | 3350 | 3340 | 3320 | 3310 | 3300 | 3260 | 3240 | 3220 |
| 225...... | 3430 | 3420 | 3410 | 3410 | 3400 | 3380 | 3370 | 3360 | 3340 | 3320 | 3280 | 3250 | 3230 |
| 200...... | 3470 | 3440 | 3430 | 3430 | 3420 | 3420 | 3410 | 3380 | 3370 | 3350 | 3300 | 3260 | 3240 |
| 175...... | 3500 | 3480 | 3480 | 3480 | 3470 | 3460 | 3430 | 3420 | 3410 | 3360 | 3310 | 3300 | 3260 |
| 160...... | 3530 | 3520 | 3500 | 3500 | 3490 | 3480 | 3470 | 3440 | 3420 | 3380 | 3350 | 3310 | 3280 |
| 150...... | 3550 | 3530 | 3540 | 3530 | 3520 | 3500 | 3490 | 3460 | 3440 | 3410 | 3360 | 3320 | 3290 |
| 140...... | 3580 | 3560 | 3550 | 3540 | 3540 | 3530 | 3530 | 3480 | 3470 | 3420 | 3370 | 3340 | 3300 |
| 130...... | 3600 | 3590 | 3580 | 3570 | 3560 | 3550 | 3550 | 3500 | 3490 | 3430 | 3380 | 3350 | .... |
| 120...... | 3620 | 3610 | 3600 | 3590 | 3590 | 3550 | 3550 | 3530 | 3500 | 3460 | 3410 | .... | .... |
| 110...... | 3650 | 3640 | 3640 | 3630 | 3620 | 3600 | 3590 | 3560 | 3530 | 3480 | .... | .... | .... |
| 100...... | 3700 | 3680 | 3670 | 3660 | 3650 | 3640 | 3610 | 3590 | 3560 | .... | .... | .... | .... |
| 95...... | 3740 | 3720 | 3690 | 3680 | 3670 | 3660 | 3620 | 3600 | .... | .... | .... | .... | .... |
| 90...... | 3770 | 3740 | 3720 | 3700 | 3690 | 3670 | 3650 | .... | .... | .... | .... | .... | .... |
| 85...... | 3790 | 3770 | 3740 | 3730 | 3710 | 3680 | .... | .... | .... | .... | .... | .... | .... |
| 80...... | 3830 | 3800 | 3770 | 3750 | 3730 | .... | .... | .... | .... | .... | .... | .... | .... |
| 75...... | 3840 | 3820 | 3780 | 3770 | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| 70...... | 3840 | 3820 | 3790 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| 65...... | 3840 | 3820 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| 60...... | 3830 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |

For $l_1$ and $l_2$ each $>142$ ft. $q = \left(3.0 + \dfrac{11400}{l_1 L}\right) 1000$



## TABLE 19

### Influence-Line Ordinates for $M$ for Girder Bridges Without Floor-beams

Values of $\frac{l_1 l_2}{L}$

Shorter Segment $l_1$

| Longer Segment $l_2$ | 5 | 10 | 15 | 20 | 25 | 30 | 35 | 40 | 45 | 50 | 55 | 60 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 250...... | 4.90 | 9.62 | 14.14 | 18.5 | 22.7 | 26.7 | 30.7 | 34.5 | 38.2 | 41.7 | 45.2 | 48.3 |
| 225...... | 4.90 | 9.62 | 14.06 | 18.4 | 22.5 | 26.5 | 30.3 | 33.9 | 37.6 | 41.0 | 44.2 | 47.4 |
| 200...... | 4.88 | 9.52 | 13.97 | 18.2 | 22.2 | 26.1 | 29.9 | 33.3 | 36.8 | 40.0 | 43.1 | 46.1 |
| 175...... | 4.85 | 9.43 | 13.83 | 17.9 | 21.9 | 25.6 | 29.2 | 32.6 | 35.8 | 38.9 | 42.0 | 44.6 |
| 160...... | 4.85 | 9.43 | 13.70 | 17.8 | 21.6 | 25.3 | 28.7 | 32.0 | 35.2 | 38.0 | 41.0 | 43.7 |
| 150...... | 4.83 | 9.35 | 13.64 | 17.6 | 21.5 | 25.0 | 28.4 | 31.6 | 34.7 | 37.6 | 40.3 | 42.9 |
| 140...... | 4.83 | 9.34 | 13.55 | 17.5 | 21.2 | 24.7 | 28.0 | 31.1 | 34.1 | 36.8 | 39.5 | 42.0 |
| 130...... | 4.81 | 9.29 | 13.44 | 17.3 | 21.0 | 24.4 | 27.6 | 30.6 | 33.4 | 36.1 | 38.6 | 41.0 |
| 120...... | 4.80 | 9.23 | 13.33 | 17.2 | 20.7 | 24.0 | 27.1 | 30.0 | 32.7 | 35.3 | 37.7 | 40.0 |
| 110...... | 4.78 | 9.17 | 13.19 | 16.9 | 20.4 | 23.6 | 26.6 | 29.3 | 31.9 | 34.4 | 36.6 | 38.7 |
| 100...... | 4.76 | 9.09 | 13.05 | 16.7 | 20.0 | 23.1 | 25.9 | 28.6 | 31.1 | 33.3 | 35.5 | 37.5 |
| 95...... | 4.75 | 9.05 | 12.95 | 16.5 | 19.8 | 22.8 | 25.6 | 28.1 | 30.6 | 32.8 | 34.8 | 36.7 |
| 90...... | 4.74 | 9.00 | 12.85 | 16.4 | 19.6 | 22.5 | 25.2 | 27.7 | 30.0 | 32.2 | 34.1 | 36.1 |
| 85...... | 4.72 | 8.94 | 12.76 | 16.2 | 19.3 | 22.2 | 24.8 | 27.2 | 29.4 | 31.5 | 33.4 | 35.2 |
| 80...... | 4.71 | 8.89 | 12.63 | 16.0 | 19.0 | 21.8 | 24.3 | 26.7 | 28.8 | 30.8 | 32.6 | 34.3 |
| 75...... | 4.69 | 8.83 | 12.50 | 15.8 | 18.8 | 21.5 | 23.9 | 26.1 | 28.1 | 30.0 | 31.8 | 33.3 |
| 70...... | 4.67 | 8.75 | 12.35 | 15.6 | 18.4 | 21.0 | 23.4 | 25.5 | 27.4 | 29.2 | 30.8 | 32.4 |
| 65...... | 4.64 | 8.67 | 12.20 | 15.3 | 18.0 | 20.5 | 22.7 | 24.8 | 26.6 | 28.3 | 29.8 | 31.2 |
| 60...... | 4.62 | 8.58 | 12.00 | 15.0 | 17.6 | 20.0 | 22.1 | 24.0 | 25.8 | 27.3 | 28.7 | 30.0 |
| 55...... | 4.58 | 8.46 | 11.79 | 14.7 | 17.2 | 19.4 | 21.4 | 23.2 | 24.8 | 26.2 | 27.5 | .... |
| 50...... | 4.55 | 8.33 | 11.53 | 14.3 | 16.7 | 18.8 | 20.6 | 22.2 | 23.7 | 25.0 | .... | .... |
| 45...... | 4.50 | 8.18 | 11.25 | 13.9 | 16.1 | 18.0 | 19.7 | 21.2 | 22.5 | .... | .... | .... |
| 40...... | 4.44 | 8.00 | 10.91 | 13.3 | 15.4 | 17.2 | 18.7 | 20.0 | .... | .... | .... | .... |
| 35...... | 4.37 | 7.78 | 10.50 | 12.7 | 14.6 | 16.2 | 17.5 | .... | .... | .... | .... | .... |
| 30...... | 4.29 | 7.50 | 10.00 | 12.0 | 13.6 | 15.0 | .... | .... | .... | .... | .... | .... |
| 25...... | 4.17 | 7.14 | 9.38 | 11.1 | 12.5 | .... | .... | .... | .... | .... | .... | .... |
| 20...... | 4.00 | 6.67 | 8.58 | 10.0 | .... | .... | .... | .... | .... | .... | .... | .... |
| 15...... | 3.75 | 6.00 | 7.50 | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| 10...... | 3.33 | 5.00 | ..... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| 5...... | 2.50 | .... | ..... | .... | .... | .... | .... | .... | .... | .... | .... | .... |

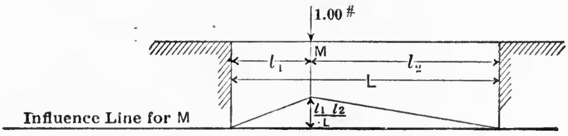
TABLE 19.—*Continued*

INFLUENCE-LINE ORDINATES FOR $M$ FOR GIRDER BRIDGES WITHOUT FLOOR-BEAMS

Values of $\frac{l_1 l_2}{L}$

SHORTER SEGMENT $l_1$

| | 65 | 70 | 75 | 80 | 85 | 90 | 95 | 100 | 110 | 120 | 130 | 140 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 250 | 51.5 | 54.6 | 57.5 | 60.6 | 63.3 | 66.2 | 69.0 | 71.4 | 76.3 | 81.3 | 85.5 | 89.3 |
| 225 | 50.5 | 53.2 | 56.2 | 58.8 | 61.7 | 64.1 | 66.7 | 69.4 | 73.5 | 78.1 | 82.0 | 86.2 |
| 200 | 49.0 | 51.8 | 54.6 | 57.1 | 59.5 | 62.1 | 64.5 | 66.8 | 70.9 | 75.2 | 78.7 | 82.0 |
| 175 | 47.2 | 50.0 | 52.4 | 54.9 | 57.1 | 59.5 | 61.7 | 63.7 | 67.6 | 71.4 | 74.6 | 78.1 |
| 160 | 46.1 | 48.5 | 51.0 | 53.2 | 55.6 | 57.5 | 59.5 | 61.7 | 64.9 | 68.5 | 71.4 | 74.6 |
| 150 | 45.2 | 47.6 | 50.0 | 52.1 | 54.3 | 56.2 | 58.1 | 59.9 | 63.3 | 66.7 | 69.4 | 72.5 |
| 140 | 44.4 | 46.7 | 49.0 | 51.0 | 52.9 | 54.6 | 56.5 | 58.5 | 61.7 | 64.9 | 67.6 | 70.0 |
| 130 | 43.3 | 45.5 | 47.6 | 49.5 | 51.6 | 53.2 | 55.0 | 56.5 | 59.5 | 62.5 | 65.0 | .... |
| 120 | 42.2 | 44.3 | 46.3 | 48.1 | 49.8 | 51.5 | 53.2 | 54.6 | 57.5 | 60.0 | .... | .... |
| 110 | 40.8 | 42.7 | 44.6 | 46.3 | 48.1 | 49.5 | 51.0 | 52.4 | 55.0 | .... | .... | .... |
| 100 | 39.4 | 41.2 | 42.9 | 44.4 | 46.1 | 47.4 | 48.8 | 50.0 | .... | .... | .... | .... |
| 95 | 38.6 | 40.3 | 42.0 | 43.5 | 44.8 | 46.3 | 47.5 | .... | .... | .... | .... | .... |
| 90 | 37.7 | 39.4 | 41.0 | 42.4 | 43.7 | 45.0 | .... | .... | .... | .... | .... | .... |
| 85 | 36.8 | 38.3 | 39.8 | 41.2 | 42.5 | .... | .... | .... | .... | .... | .... | .... |
| 80 | 35.8 | 37.3 | 38.7 | 40.0 | .... | .... | .... | .... | .... | .... | .... | .... |
| 75 | 34.8 | 36.2 | 37.5 | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| 70 | 33.8 | 35.0 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |
| 65 | 32.5 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... |



# TABLE 20

## Reciprocals of Influence-Line Ordinates for $M$ for Girder Bridges Without Floor-Beams

Values of $\frac{L}{l_1 l_2}$

Shorter Segment $l_1$

| Longer Segment $l_2$ | 5 | 10 | 15 | 20 | 25 | 30 | 35 | 40 | 45 | 50 | 55 | 60 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 250 | .204 | .104 | .0707 | .0540 | .0440 | .0374 | .0326 | .0290 | .0262 | .0240 | .0221 | .0207 |
| 225 | .204 | .104 | .0711 | .0544 | .0444 | .0378 | .0330 | .0295 | .0266 | .0244 | .0226 | .0211 |
| 200 | .205 | .105 | .0716 | .0550 | .0450 | .0383 | .0335 | .0300 | .0272 | .0250 | .0232 | .0217 |
| 175 | .206 | .106 | .0723 | .0558 | .0457 | .0390 | .0342 | .0307 | .0279 | .0257 | .0238 | .0224 |
| 160 | .206 | .106 | .0730 | .0562 | .0462 | .0396 | .0348 | .0313 | .0284 | .0263 | .0244 | .0229 |
| 150 | .207 | .107 | .0733 | .0567 | .0466 | .0400 | .0352 | .0317 | .0288 | .0266 | .0248 | .0233 |
| 140 | .207 | .107 | .0738 | .0571 | .0472 | .0405 | .0357 | .0321 | .0293 | .0271 | .0253 | .0238 |
| 130 | .208 | .108 | .0744 | .0577 | .0477 | .0410 | .0363 | .0327 | .0299 | .0277 | .0259 | .0244 |
| 120 | .208 | .108 | .0750 | .0583 | .0483 | .0417 | .0369 | .0333 | .0306 | .0283 | .0265 | .0250 |
| 110 | .209 | .109 | .0758 | .0591 | .0491 | .0424 | .0376 | .0341 | .0314 | .0291 | .0273 | .0258 |
| 100 | .210 | .110 | .0766 | .0600 | .0500 | .0433 | .0386 | .0350 | .0322 | .0300 | .0282 | .0267 |
| 95 | .211 | .111 | .0772 | .0605 | .0505 | .0438 | .0391 | .0355 | .0327 | .0305 | .0287 | .0272 |
| 90 | .211 | .111 | .0778 | .0611 | .0511 | .0444 | .0397 | .0361 | .0333 | .0311 | .0293 | .0277 |
| 85 | .212 | .112 | .0784 | .0618 | .0517 | .0451 | .0403 | .0368 | .0340 | .0318 | .0299 | .0284 |
| 80 | .213 | .113 | .0792 | .0625 | .0525 | .0458 | .0411 | .0375 | .0347 | .0325 | .0307 | .0292 |
| 75 | .213 | .113 | .0800 | .0633 | .0533 | .0466 | .0419 | .0383 | .0356 | .0333 | .0315 | .0300 |
| 70 | .214 | .114 | .0810 | .0643 | .0543 | .0476 | .0428 | .0393 | .0365 | .0343 | .0325 | .0309 |
| 65 | .215 | .115 | .0820 | .0654 | .0554 | .0487 | .0440 | .0404 | .0376 | .0353 | .0336 | .0321 |
| 60 | .217 | .117 | .0833 | .0666 | .0567 | .0500 | .0452 | .0417 | .0388 | .0366 | .0348 | .0333 |
| 55 | .218 | .118 | .0848 | .0682 | .0582 | .0515 | .0467 | .0432 | .0404 | .0382 | .0364 | ..... |
| 50 | .220 | .120 | .0867 | .0700 | .0600 | .0533 | .0486 | .0450 | .0422 | .0400 | ..... | ..... |
| 45 | .222 | .122 | .0889 | .0722 | .0622 | .0555 | .0508 | .0472 | .0444 | ..... | ..... | ..... |
| 40 | .225 | .125 | .0917 | .0750 | .0650 | .0583 | .0536 | .0500 | ..... | ..... | ..... | ..... |
| 35 | .229 | .129 | .0952 | .0786 | .0686 | .0619 | .0571 | ..... | ..... | ..... | ..... | ..... |
| 30 | .233 | .133 | .1000 | .0833 | .0733 | .0666 | ..... | ..... | ..... | ..... | ..... | ..... |
| 25 | .240 | .140 | .1066 | .0900 | .0800 | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 20 | .250 | .150 | .1166 | .1000 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 15 | .267 | .167 | .1333 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 10 | .300 | .200 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 5 | .400 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |

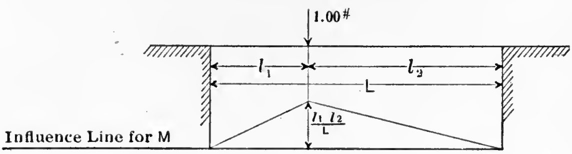
## TABLE 20.—*Continued*

### Reciprocals of Influence-Line Ordinates for $M$ for Girder Bridges Without Floor-Beams

Values of $\frac{L}{l_1 l_2}$

Shorter Segment $l_1$

| | 65 | 70 | 75 | 80 | 85 | 90 | 95 | 100 | 110 | 120 | 130 | 140 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 250 | .0194 | .0183 | .0174 | .0165 | .0158 | .0151 | .0145 | .0140 | .0131 | .0123 | .0117 | .0112 |
| 225 | .0198 | .0188 | .0178 | .0170 | .0162 | .0156 | .0150 | .0144 | .0136 | .0128 | .0122 | .0116 |
| 200 | .0204 | .0193 | .0183 | .0175 | .0168 | .0161 | .0155 | .0150 | .0141 | .0133 | .0127 | .0122 |
| 175 | .0212 | .0200 | .0191 | .0182 | .0175 | .0168 | .0162 | .0157 | .0148 | .0140 | .0134 | .0128 |
| 160 | .0217 | .0206 | .0196 | .0188 | .0180 | .0174 | .0168 | .0162 | .0154 | .0146 | .0140 | .0134 |
| 150 | .0221 | .0210 | .0200 | .0192 | .0184 | .0178 | .0172 | .0167 | .0158 | .0150 | .0144 | .0138 |
| 140 | .0225 | .0214 | .0204 | .0196 | .0189 | .0183 | .0177 | .0171 | .0162 | .0154 | .0148 | .0143 |
| 130 | .0231 | .0220 | .0210 | .0202 | .0194 | .0188 | .0182 | .0177 | .0168 | .0160 | .0154 | ..... |
| 120 | .0237 | .0226 | .0216 | .0208 | .0201 | .0194 | .0188 | .0183 | .0174 | .0167 | ..... | ..... |
| 110 | .0245 | .0234 | .0224 | .0216 | .0208 | .0202 | .0196 | .0191 | .0182 | ..... | ..... | ..... |
| 100 | .0254 | .0243 | .0233 | .0225 | .0217 | .0211 | .0205 | .0200 | ..... | ..... | ..... | ..... |
| 95 | .0259 | .0248 | .0238 | .0230 | .0223 | .0216 | .0211 | ..... | ..... | ..... | ..... | ..... |
| 90 | .0265 | .0254 | .0244 | .0236 | .0229 | .0222 | ..... | ..... | ..... | ..... | ..... | ..... |
| 85 | .0272 | .0261 | .0251 | .0243 | .0235 | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 80 | .0279 | .0268 | .0258 | .0250 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 75 | .0287 | .0276 | .0266 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 70 | .0296 | .0286 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 65 | .0307 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |



## TABLE 21

### Bending Moments in Beams Due to Uniform Load of 1 Pound per Lineal Foot

Values in Foot-pounds

Values equal $\frac{l_1 l_2}{2}$ = Area of Influence Line for $M$

Shorter Segment $l_1$

| Longer Segment $l_2$ | 5 | 10 | 15 | 20 | 25 | 30 | 35 | 40 | 45 | 50 | 55 | 60 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 250 | 625 | 1250 | 1875 | 2500 | 3125 | 3750 | 4375 | 5000 | 5625 | 6250 | 6875 | 7500 |
| 225 | 562.5 | 1125 | 1687.5 | 2250 | 2812.5 | 3375 | 3937.5 | 4500 | 5062.5 | 5625 | 6187.5 | 6750 |
| 200 | 500 | 1000 | 1500 | 2000 | 2500 | 3000 | 3500 | 4000 | 4500 | 5000 | 5500 | 6000 |
| 175 | 437.5 | 875 | 1312.5 | 1750 | 2187.5 | 2625 | 3062.5 | 3500 | 3937.5 | 4375 | 4812.5 | 5250 |
| 160 | 400 | 800 | 1200 | 1600 | 2000 | 2400 | 2800 | 3200 | 3600 | 4000 | 4400 | 4800 |
| 150 | 375 | 750 | 1125 | 1500 | 1875 | 2250 | 2625 | 3000 | 3375 | 3750 | 4125 | 4500 |
| 140 | 350 | 700 | 1050 | 1400 | 1750 | 2100 | 2450 | 2800 | 3150 | 3500 | 3850 | 4200 |
| 130 | 325 | 650 | 975 | 1300 | 1625 | 1950 | 2275 | 2600 | 2925 | 3250 | 3575 | 3900 |
| 120 | 300 | 600 | 900 | 1200 | 1500 | 1800 | 2100 | 2400 | 2700 | 3000 | 3300 | 3600 |
| 110 | 275 | 550 | 825 | 1100 | 1375 | 1650 | 1925 | 2200 | 2475 | 2750 | 3025 | 3300 |
| 100 | 250 | 500 | 750 | 1000 | 1250 | 1500 | 1750 | 2000 | 2250 | 2500 | 2750 | 3000 |
| 95 | 237.5 | 475 | 712.5 | 950 | 1187.5 | 1425 | 1662.5 | 1900 | 2137.5 | 2375 | 2612.5 | 2850 |
| 90 | 225 | 450 | 675 | 900 | 1125 | 1350 | 1575 | 1800 | 2025 | 2250 | 2475 | 2700 |
| 85 | 212.5 | 425 | 637.5 | 850 | 1062.5 | 1275 | 1487.5 | 1700 | 1912.5 | 2125 | 2337.5 | 2550 |
| 80 | 200 | 400 | 600 | 800 | 1000 | 1200 | 1400 | 1600 | 1800 | 2000 | 2200 | 2400 |
| 75 | 187.5 | 375 | 562.5 | 750 | 937.5 | 1125 | 1312.5 | 1500 | 1687.5 | 1875 | 2062.5 | 2250 |
| 70 | 175 | 350 | 525 | 700 | 875 | 1050 | 1225 | 1400 | 1575 | 1750 | 1925 | 2100 |
| 65 | 162.5 | 325 | 487.5 | 650 | 812.5 | 975 | 1137.5 | 1300 | 1462.5 | 1625 | 1787.5 | 1950 |
| 60 | 150 | 300 | 450 | 600 | 750 | 900 | 1050 | 1200 | 1350 | 1500 | 1650 | 1800 |
| 55 | 137.5 | 275 | 412.5 | 550 | 687.5 | 825 | 962.5 | 1100 | 1237.5 | 1375 | 1512.5 | ... |
| 50 | 125 | 250 | 375 | 500 | 625 | 750 | 875 | 1000 | 1125 | 1250 | ... | ... |
| 45 | 112.5 | 225 | 337.5 | 450 | 562.5 | 675 | 787.5 | 900 | 1012.5 | ... | ... | ... |
| 40 | 100 | 200 | 300 | 400 | 500 | 600 | 700 | 800 | ... | ... | ... | ... |
| 35 | 87.5 | 175 | 262.5 | 350 | 437.5 | 525 | 612.5 | ... | ... | ... | ... | ... |
| 30 | 75.0 | 150 | 225 | 300 | 375 | 450 | ... | ... | ... | ... | ... | ... |
| 25 | 62.5 | 125 | 187.5 | 250 | 312.5 | ... | ... | ... | ... | ... | ... | ... |
| 20 | 50.0 | 100 | 150 | 200 | ... | ... | ... | ... | ... | ... | ... | ... |
| 15 | 37.5 | 75 | 112.5 | ... | ... | ... | ... | ... | ... | ... | ... | ... |
| 10 | 25.0 | 50 | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... |
| 5 | 12.5 | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... |

## TABLE 21.—*Continued*

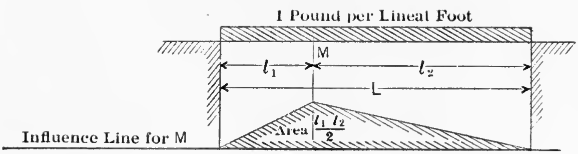
### Bending Moments in Beams Due to Uniform Load of 1 Pound per Lineal Foot

Values in Foot-pounds

Values equal $\frac{l_1 l_2}{2}$ = Area of Influence Line for $M$

Shorter Segment $l_1$

| | 65 | 70 | 75 | 80 | 85 | 90 | 95 | 100 | 110 | 120 | 130 | 140 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 250 | 8125 | 8750 | 9375 | 10000 | 10625. | 11250 | 11875 | 12500 | 13750 | 15000 | 16250 | 17500 |
| 225 | 7312.5 | 7875 | 8437.5 | 9000 | 9562.5 | 10125 | 10687.5 | 11250 | 12375 | 13500 | 14625 | 15750 |
| 200 | 6500 | 7000 | 7500 | 8000 | 8500 | 9000 | 9500 | 10000 | 11000 | 12000 | 13000 | 14000 |
| 175 | 5687.5 | 6125 | 6562.5 | 7000 | 7437.5 | 7875 | 8312.5 | 8750 | 9625 | 10500 | 11375 | 12225 |
| 160 | 5200 | 5600 | 6000 | 6400 | 6800 | 7200 | 7600 | 8000 | 8800 | 9600 | 10400 | 11200 |
| 150 | 4875 | 5250 | 5625 | 6000 | 6375 | 6750 | 7125 | 7500 | 8250 | 9000 | 9750 | 10500 |
| 140 | 4550 | 4900 | 5250 | 5600 | 5950 | 6300 | 6650 | 7000 | 7700 | 8400 | 9100 | 9800 |
| 130 | 4225 | 4550 | 4875 | 5200 | 5525 | 5850 | 6175 | 6500 | 7150 | 7800 | 8450 | ..... |
| 120 | 3900 | 4200 | 4500 | 4800 | 5100 | 5400 | 5700 | 6000 | 6600 | 7200 | ..... | ..... |
| 110 | 3575 | 3850 | 4125 | 4400 | 4675 | 4950 | 5225 | 5500 | 6050 | ..... | ..... | ..... |
| 100 | 3250 | 3500 | 3750 | 4000 | 4250 | 4500 | 4750 | 5000 | ..... | ..... | ..... | ..... |
| 95 | 3087.5 | 3325 | 3562.5 | 3800 | 4037.5 | 4275 | 4512.5 | ..... | ..... | ..... | ..... | ..... |
| 90 | 2925 | 3150 | 3375 | 3600 | 3825 | 4050 | ..... | ..... | ..... | ..... | ..... | ..... |
| 85 | 2762.5 | 2975 | 3187.5 | 3400 | 3612.5 | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 80 | 2600 | 2800 | 3000 | 3200 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 75 | 2437.5 | 2625 | 2812.5 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 70 | 2275 | 2450 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 65 | 2112.5 | .... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |



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