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LIVE-LOAD STRESSES

IN

RAILWAY BRIDGES

WITH

FORMULAS AND TABLES

BY

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PREFACE

STRESSES caused by moving concentrated loads are treated in this book by the combined use of influence lines and algebraic methods. The influence line is connected by this treatment with tables of moment sums and load sums in a new and entirely practical manner.

The heart of the text is contained in equations (7) and (8). These give an easy and exact solution of the maximum live-load stresses in any structure whose influence lines can be drawn, replacing, for the more complicated structures, such as cantilever and swing bridges, arches, etc., the old method of placing the wheel loading by trial and scaling the influence-line ordinates under the loads.

A second feature of the text is the application of equations (7) and (8) to the simpler structures, such as girder bridges (with and without panels), pier reactions, and Pratt trusses (with inclined and horizontal chords), in which these equations are transformed and simplified to meet the requirements of these ordinary cases. This leads to a series of simple formulas to meet the needs of every-day designing. To illustrate the application of these formulas, fully worked-out examples are given.

The text is supplemented by a very complete set of tables, the usefulness of which is at once apparent. The greater part of the matter in these tables is new. A table similar to Table 3 was made by Mr. Josiah Gibson, C.E., and published in the *Engineering News*, June 21, 1906; and a table similar to Table 11 is given by Mr. J. P. J. Williams in the *Engineering News* of Oct. 1, 1914. Tables similar to Tables 6, 8, and 9 are found in the "Structural Engineers' Handbook" by Dean Milo S. Ketchum and in the "Design of Steel Bridges" by Mr. F. C. Kunz.

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G. E. B.

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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 repre-

sending 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

$$Z = w_1 z_1 + w_2 z_2 + w_3 z_3 + \dots = \Sigma wz \dots (1)$$

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:

$Z = \text{Sum of ordinate-load products.}$

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_s 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,

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

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

$$Z = \sum wz = w \sum z \quad \dots \dots \dots (3)$$

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

$$Z = z(w_1 + w_2 + \dots) = z \sum w = zW \quad \dots \dots (4)$$

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

$$Z = C_a M_a \dots \dots \dots (5)$$

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

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

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,

$$w_n z_n = C_a w_n x_n. \quad \dots \quad (A)$$

Summing up all of the ordinate-load products,

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

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

$$w_n dz_n = C_a w_n dx_n.$$

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

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

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

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_1L_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.

$$\begin{array}{ll}
 s_1 = \frac{0 - z_2}{l_1} (+) & C_1 = 0 - s_1 (-) \\
 s_2 = \frac{z_2 - z_3}{l_2} (-) & C_2 = s_1 - s_2 (+) \\
 s_3 = \frac{z_3 - z_4}{l_3} (+) & C_3 = s_2 - s_3 (-) \\
 s_4 = \frac{z_4 - 0}{l_4} (+) & C_4 = s_3 - s_4 (+) \\
 & C_5 = s_4 - 0 (+)
 \end{array}$$

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.

$$\sum C = 0. \quad (6)$$

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	$\frac{dfe}{cge} = C_1 M_1$	(-)
" " " "	$\frac{cge}{eha} = C_2 M_2$	(+)
" " " "	$\frac{eha}{akb} = C_3 M_3$	(-)
" " " "	$\frac{akb}{bmd} = C_4 M_4$	(+)
" " " "	$\frac{bmd}{fom} = C_5 M_5$	(+)

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

$$S = C_1 M_1 + C_2 M_2 + \dots = \sum CM. \quad (7)$$

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

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

But by formula (5a) this becomes

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

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,

$$\begin{array}{rcccccl} & & & & & \text{Ordinate-load products in } \boxed{rst} = \\ & & & & & \text{---} \quad \text{---} \quad \text{---} \quad \text{---} \quad \boxed{atc} \\ & & & & & \text{---} \quad \text{---} \quad \text{---} \quad \text{---} \quad \boxed{arb} \\ & & & & & \text{---} \quad \text{---} \quad \text{---} \quad \text{---} \quad \boxed{brsc}. \end{array}$$

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,

$$\begin{array}{ll} \text{At 1, } 14' \text{ from } w_1, M_1 = 350.0^{K_1} & W_1 = 62.50^K \\ \text{At 2, } 24' \text{ from } w_1, M_2 = 1150.0 & W_2 = 112.50 \\ \text{At 3, } 54' \text{ from } w_1, M_3 = 5435.0 & W_3 = 177.50 \end{array}$$

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),

$$R_2 = \text{Ordinate-load products in } (d\text{v}x\text{e} - \boxed{d\text{v}f} + \boxed{f\text{u}e})$$

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:

$$M = \text{Ordinate-load products in } (\boxed{g\text{b}h} - \boxed{g\text{a}k} + \boxed{k\text{z}h}).$$

Or

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

Formula (10) readily follows, likewise, from the general formula (7), $S = C_1 M_1 + C_2 M_2 + C_3 M_3 = \Sigma C M$.

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

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

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

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

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):

$S = \text{Ordinate-load products in}$

$$(\boxed{mfq} - mden - \boxed{ncq})$$

Or

$$S = \frac{1}{L} M_2 - W_2 - \frac{1}{L} M_1 = \frac{M_2 - M_1}{L} - W_2 \quad (12)$$

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 $(W_2 - W_1)$. 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 $R_1 = \frac{P_2 \bar{x}}{L}$ it follows that $\bar{x} = \frac{R_1 L}{P_2}$. Substitute the value of R_1 from formula (9), and the value $(W_2 - W_1)$ for P_2 .

$$\bar{x} = \frac{M_2 - M_1 - L W_1}{W_2 - W_1} \quad \dots \dots (13)$$

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

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

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), $R_1 = \frac{M_3 - M_1}{L} - W_1$. 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.

$$S = \frac{M_2 - M_1}{L} - W_2$$

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

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),

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

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

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

No maximum.

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

w_2 at $\frac{1}{4}$ point.

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

Maximum.

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

w_3 at $\frac{1}{4}$ point.

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

Maximum.

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

w_4 at $\frac{1}{4}$ point.

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

No maximum.

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

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

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

M for w_2 at quarter point,

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

M for w_3 at quarter point,

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

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.

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

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

w_2 at centre,

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

No maximum.

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

w_1 at centre,

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

Maximum.

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

w_3 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_1 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 $\frac{20'.00 - 19'.58}{2} = 0'.21$.

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 =

$$\begin{aligned} M &= \frac{l_1}{L} M_3 + \frac{l_2}{L} M_1 - M_2 \\ &= \frac{19.79}{40} (2869.2) + 0 - 600 = 819.54 \text{ Kip feet.} \end{aligned}$$

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

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

Or,

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

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,

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

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

For equal spans,

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

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

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

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

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

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 .

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

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 .

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

w_2 at pier.

$$R = \frac{2838.75}{30} + \frac{0}{10} - \frac{40}{10 \times 30} (100) = 81^4.$$

w_3 at pier.

$$R = \frac{3563.75}{30} + \frac{37.5}{10} - \frac{40}{10 \times 30} (287.5) = 84^4.$$

The latter value of 84^4 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),

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

w_2 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).

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

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

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

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

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)

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

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

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

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

$$R_1 = \frac{27651 - 100}{120} - 12.5 = 217.1^1$$

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.

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

Maximum.

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

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).

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

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

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.

PLEASE RETURN TO
DEPT. of APPLIED MECHANICS.

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 $\frac{dS}{dx} = \Sigma CW$ 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.

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

$$C_1 = -\frac{k}{nv} - \frac{m}{nv} = -\frac{1}{v}$$

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

Therefore, for the position of the live load advanced beyond the limits of the span, the general formula for S_1 is

$$S_1 = \left(\frac{m}{nv}\right)M_1 - \left(\frac{1}{v}\right)M_2 + \left(\frac{k}{nv}\right)M_3.$$

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_1 . The usual formula will therefore not contain the term M_3 , since this will be zero; thus,

$$S_1 = \left(\frac{m}{nv}\right)M_1 - \left(\frac{1}{v}\right)M_2 \quad \dots \dots \dots (21)$$

Inasmuch as the horizontal component of the stress S_1 in an inclined top chord member or end post equals the stress S_2 in a corresponding lower chord member, the stress S_2 in any top chord member or end post may be found by

$$S_2 = \frac{i}{p} \cdot S_1 \quad \dots \dots \dots (22)$$

In Fig. 8 is shown the influence line for the stress S_1 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,

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

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:

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

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

$$S_2 = \left(\frac{ta}{cbL}\right)M_1 - \left(\frac{t}{bp}\right)M_1 + \left(\frac{t}{cp}\right)M_2$$

$$\text{Or, letting } M_2 = \left(M_1 - \frac{b}{c}M_1\right),$$

$$S_2 = \left(\frac{ta}{cbL}\right)M_1 - \frac{t}{bp}\left(M_1 - \frac{b}{c}M_1\right) = \left(\frac{ta}{cbL}\right)M_1 - \left(\frac{t}{bp}\right)M_1 \quad (22)$$

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_o 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 $\frac{2}{3}$ 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 =

$$S_3 = \left(\frac{m}{nv}\right)M_1 - \left(\frac{1}{v}\right)M_2 \dots \dots \dots (21)$$

Stress in inclined end post = $S_4 = \frac{i}{p} S_3 \dots \dots \dots (22)$

Stress in vertical post = $S_5 = \left(\frac{1}{l}\right)M_1 - \left(\frac{1}{p}\right)M_2 \dots \dots (23)$

Stress in inclined web member =

$$S_1 = \left(\frac{t}{cL}\right)M_1 - \left(\frac{t}{cp}\right)M_2 = \frac{t}{c} S_5 \dots \dots \dots (24)$$

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

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

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

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

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 $\frac{1}{2}$ 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 $\left(\frac{G}{H} W_1 - W_2\right)$

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 $S = GM_1 - HM_1$ 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 ₁	M ₂	GM ₁	HM ₁	S	L ₁	$\frac{300}{L_1 + 300}$	I	DL	Total
EF	00373	0385	3 @	333970	287	127	11	-116	143	.677	-78	-40	-234
ED	00481	0442	3 @	246255	287	223	13	+210	169	.640	+134	+83	+497
GH	00405	0385	2 @	421531	100	87	4	-83	112	.728	-60	-15	-128
GF	00500	0450	3 @	333970	287	170	13	+157	143	.677	+106	+48	+311
IJ	00480	0385	2 @	512940	100	62	4	-58	86	.777	-45	+7	-
IH	00580	0466	3 @	423375	287	136	13	+123	117	.719	+88	+21	+232
JK	00580	0466	2 @	512940	100	75	5	+70	86	.777	+54	-21	-
ML	00777	0493	2 @	6550	100	51	5	+46	60	.833	+38	-50	-
NO	01030	0496	2 @	72307	100	24	5	-19	34	.898	-17	+83	No
AC-AD	00390	0312	4 @	163111	600	247	19	+228	200	.600	+137	+101	+466
BC								-362			-217	-160	-739
AF	00695	0278	7 @	259095	2694	410	75	+335	193	.608	+203	+154	+602
BE								-336			-206	-156	-761
AH	00985	0263	11 @	359661	7310	587	192	+396	194	.607	+239	+181	+815
BG								-396			-240	-181	-817
BI	01315	0263	13 @	450901	9685	670	232	-418	178	.627	-292	-194	-874
CD	0385	0770	4 @	13725	600	144	46	+98	44	.872	+80	+25	+209

Post at	Mem.	M ₁	M ₂	S	LD	K	M ₁	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.

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

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. $L_1 = 143'$.

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

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 $S_1 = \left(\frac{ta}{cbL}\right) M_1 - \left(\frac{t}{bp}\right) M_1 \dots \dots (24)$

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_2 at panel point 2. Use Table 2. $L_1 = 169'$.

$$\left(\frac{G}{H} W_1 - W_2\right) = \frac{.00481}{.0442} (505.0) - \frac{37.5}{62.5} = \frac{+}{-}$$

Therefore w_2 at 2 gives a maximum.

$$S = GM_1 - HM_2 = .00481(46255) - .0442(287.5) \\ = 223 - 13 = 210^k.$$

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

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

Inclined Web Member ML.

Formula $S_2 = \left(\frac{ta}{cbL}\right) M_1 - \left(\frac{t}{bp}\right) M_1 \dots \dots (24)$

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. $L_1 = 60'$.

$$\left(\frac{G}{H} W_1 - W_2\right) = \frac{.00777}{.0493} (190) - \frac{12.5}{37.5} = \frac{+}{-}$$

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.

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

Refer to Fig. 11 for definition of dimensions.

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

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

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

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

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.

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

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

Lower Chord Member AH.

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

Refer to Fig. 11 for definition of dimensions.

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

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

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

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

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 $S_4 = \frac{i}{p} S_3 \dots \dots \dots (22)$

$$S_4 = \frac{26.08}{26} (395) = 396^k.$$

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

Counter-Tension in Post at Panel Point 5.

Formulas

$$S_2 = \text{Stress } JK = \left(\frac{ta}{cbL} \right) M_4 - \left(\frac{t}{bp} \right) \left(M_3 - \frac{b}{c} M_2 \right) \\ = \left(\frac{ta}{cbL} \right) M_4 - \left(\frac{t}{bp} \right) M_4 \dots \dots \dots (25)$$

T = tension in post.

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

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

$$M_4 = 22261$$

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

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

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

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.

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

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

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

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

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

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

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

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

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

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

Fig. 14.

Mem.	G	H	Wheel	M ₁	M ₂	S
CD	.0400	.0800	4 @ 1	3564	600	95
EF	.00667	.0400	3 " 3	13520	287	79
FG	.00667	.0400	2 " 4	6170	100	106
GH	.00667	.0400	2 " 5	2179	100	37
HI	.00894	.0536	3 " 2	21895	287	50
JK	.00894	.0536	4 " 1	33970	600	14
DE	.00894	.0536	7 " 2	31375	2994	181
BC	.00595	.0357	12 " 3	34411	8385	272
AC = AD	.01190	.0357				181
AF = BE	.01785	.0357				278
BG						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 $abcda$ 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_5
$U_1U_2 = \dots\dots\dots$	- .403	- .223	- .045	+ .130	+ .301
$U_1U_3 = \dots\dots\dots$	- .417	- .833	- .286	+ .262	+ .477
$U_2U_3 = \dots\dots\dots$	- .378	- .756	- 1.135	+ .189	+ .757
$U_3U_4 = \dots\dots\dots$	- .171	- .342	- .513	- .685	+ .548
$L_1L_2 = \dots\dots\dots$	- .295	- .590	- .885	- 1.180	- 1.182
$L_1L_3 = \dots\dots\dots$	+ .221	- .264	- .740	- 1.224	- 1.302
$L_2L_3 = \dots\dots\dots$	+ .217	+ .434	- .408	- 1.248	- 1.484
$L_3L_4 = \dots\dots\dots$	+ .164	+ .328	+ .491	- 1.086	- 1.674
$L_4L_5 = \dots\dots\dots$	- .048	- .096	- .145	- .193	- 1.420
$U_1L_1 = \dots\dots\dots$	- .692	- .384	- .075	+ .234	+ .345
$U_2L_2 = \dots\dots\dots$	- 1.014	- .632	- .253	+ .129	+ .287
$U_3L_3 = \dots\dots\dots$	+ .022	- .955	- .490	- .043	+ .165
$U_4L_4 = \dots\dots\dots$	+ .075	+ .150	- .775	- .317	- .076
$U_5L_5 = \dots\dots\dots$	+ .114	+ .226	+ .342	- .345	- .364
$U_1L_2 = \dots\dots\dots$	+ .800	+ .441	+ .085	- .270	- .400
$U_2L_1 = \dots\dots\dots$	+ .019	+ .878	+ .350	- .180	- .308
$U_3L_2 = \dots\dots\dots$	- .044	- .088	+ .086	+ .086	- .324
$U_4L_2 = \dots\dots\dots$	- .221	- .442	- .662	+ .928	+ .724
$U_5L_2 = \dots\dots\dots$	- .206	- .412	- .617	- .823	+ .657
$H \dots\dots\dots$	0.2187	0.4375	0.6562	0.8750	0.8750
$V \dots\dots\dots$	0.8889	0.7777	0.6666	0.5555	0.4444
$\theta \dots\dots\dots$	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 θ 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.

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.

$$C_1 = 0 - (.0105) = -.0105$$

$$C_2 = .0105 - (-.0758) = +.0863$$

$$C_3 = -.0758 - (.0336) = -.1094$$

$$C_4 = .0336 - (.0027) = +.0309$$

$$C_5 = .0027 - 0 = +.0027$$

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_3 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,

$$S = qA_s = (3.080) (27.2) = 84^k.$$

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

Compression U_3L_1 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 $A_s = 23.7$.

Therefore,

$$S = qA_s = (2.870) (23.7) = 68^k.$$

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.

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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 ,

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

The area of this influence line is

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

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

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

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

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

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

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

Substituting for R in equation (D),

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

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 formulae that were used in succession are

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

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

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

An example of the use of equivalent uniform loads has already been given in Art. 8. The general formula $S = qA_s$ 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

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

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

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 *E40* 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 XL.

SUMMARY OF FORMULAS.

Art. 1.

[illegible]

$$Z = qA, \quad (2)$$

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

$$Z = z\Sigma W = zW. \quad (4)$$

Art. 2.

$$Z = \sum w_n z_n = C_0 \sum w_n r_n = C_0 M_0 \dots \dots (5)$$

$$\frac{dZ}{dx} = C_e W_e = \frac{d(C_e M_e)}{dx} = \frac{C_e dM_e}{dx} \dots \dots (5a)$$

Art. 3.

[illegible]

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

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

Art. 4. Girder Bridge without Panels.

End reactions.

$$R_1 = \frac{M_2 - M_1}{L} - W_1 \quad (9)$$

$$R_2 = W_2 - \frac{M_2 - M_1}{L} \quad (9a)$$

Bending moment for unequal segments l_1 and l_2

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

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

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

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

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

Shear at any section.

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

Location of centre of gravity of loading on span.

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

When $M_1 = 0$,

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

Art. 5. Pier Reaction.

For unequal spans l_1 and l_2 .

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

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

For equal spans l_1 and l_2 equal to l .

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

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

Relation between R and M ,

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

Art. 6. Girder Bridge with Panels.

Shear in end panel; general case.

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

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

Shear in intermediate panel; general case.

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

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

Shear in intermediate panel; usual case.

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

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

Art. 7. Through Pratt Truss with Inclined Chord.

Stress in hanger. Use formulas (14a) and (15a).

Stress in any horizontal chord member; usual case.

$$S_4 = \left(\frac{m}{nv} \right) M_4 - \left(\frac{1}{v} \right) M_3 \quad \dots \quad (21)$$

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

$$S_4 = \left(\frac{i}{p} \right) S_3 \quad \dots \quad (22)$$

Compression in vertical post; usual case.

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

Stresses in inclined web members including counter; usual case.

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

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.

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

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

$$S = GM_4 + HM_3 \quad \dots \dots \dots (27)$$

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

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

When S in formulas (21), (23), or (24) is a maximum

$$\left(\frac{G}{H}W_4 - W_3\right) \text{ passes through zero.}$$

Through Pratt Truss—Parallel Chords.

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

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

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

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

$$\text{Stress in end post} = S_4 = \frac{S_3}{p} \quad (22)$$

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

$$S = G \cdot M_1 - H \cdot M_2 \quad (27)$$

and their rate of variation is

$$\frac{dS}{dx} = H \left(\frac{G}{H} W_1 - W_2 \right) \quad (28)$$

G and H are the coefficients of M_1 and M_2 in equations (21), (29), and (30), respectively.

When S in formulas (21), (29), or (30) is a maximum, $\left(\frac{G}{H} W_1 - W_2 \right)$ passes through zero.

Art. 9. Equivalent Uniform Loads.

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

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

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

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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'					COOPER'S E40. 50'-100'				
Length	Wheel	Load	Load Sums	Moment Sums	Length	Wheel	Load	Load Sums	Moment Sums
0	w. 1	10	10	0	50		..	...	3780
1		..	...	10	51		..	...	3922
2		..	...	20	52		..	...	4064
3		..	...	30	53		..	...	4206
4		..	...	40	54		..	...	4348
5		..	...	50	55		..	...	4490
6		..	...	60	56	w. 10	10	152	4632
7		..	...	70	57		..	...	4784
8	w. 2	20	30	80	58		..	...	4936
9		..	...	110	59		..	...	5088
10		..	...	140	60		..	...	5240
11		..	...	170	61		..	...	5392
12		..	...	200	62		..	...	5544
13	w. 3	20	50	230	63		..	...	5696
14		..	...	280	64	w. 11	20	172	5848
15		..	...	330	65		..	...	6020
16		..	...	380	66		..	...	6192
17		..	...	430	67		..	...	6364
18	w. 4	20	70	480	68		..	...	6536
19		..	...	550	69	w. 12	20	192	6708
20		..	...	620	70		..	...	6900
21		..	...	690	71		..	...	7092
22		..	...	760	72		..	...	7284
23	w. 5	20	90	830	73		..	...	7476
24		..	...	920	74	w. 13	20	212	7668
25		..	...	1010	75		..	...	7880
26		..	...	1100	76		..	...	8092
27		..	...	1190	77		..	...	8304
28		..	...	1280	78		..	...	8516
29		..	...	1370	79	w. 14	20	232	8728
30		..	...	1460	80		..	...	8960
31		..	...	1550	81		..	...	9192
32	w. 6	13	103	1640	82		..	...	9424
33		..	...	1743	83		..	...	9656
34		..	...	1846	84		..	...	9888
35		..	...	1949	85		..	...	10120
36		..	...	2052	86		..	...	10352
37	w. 7	13	116	2155	87		..	...	10584
38		..	...	2271	88	w. 15	13	245	10816
39		..	...	2387	89		..	...	11061
40		..	...	2503	90		..	...	11306
41		..	...	2619	91		..	...	11551
42		..	...	2735	92		..	...	11796
43	w. 8	13	129	2851	93	w. 16	13	258	12041
44		..	...	2980	94		..	...	12299
45		..	...	3109	95		..	...	12557
46		..	...	3238	96		..	...	12815
47		..	...	3367	97		..	...	13073
48	w. 9	13	142	3496	98		..	...	13331
49		..	...	3638	99	w. 17	13	271	13589
50		..	...	3780	100		..	...	13860

LIVE-LOAD STRESSES

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COOPER'S E40. 100'-150'

COOPER'S E40. 150'-200'

Length	Wheel	Load	Load Sum	Moment Sum	Length	Load	Load Sum	Moment Sum
100				13860	150		266	29260
101				14131	151		268	30056
102				14402	152		270	30425
103				14673	153		272	30796
104	w. 18	13	281	14944	154		274	31169
105				15228	155		276	31544
106				15512	156		278	31921
107				15796	157		280	32300
108				16080	158		282	32681
109			284	16364	159		284	33064
110			286	16649	160		286	33448
111			288	16936	161		288	33836
112			290	17225	162		290	34225
113			292	17516	163		292	34616
114			294	17809	164		294	35009
115			296	18104	165		296	35404
116			298	18401	166		298	35801
117			300	18700	167		300	36200
118			302	19001	168		302	36601
119			304	19304	169		304	37004
120		Uniform Load = 2,000 pounds per foot	306	19609	170		306	37409
121			308	19916	171		308	37816
122			310	20225	172		310	38225
123			312	20536	173		312	38636
124			314	20849	174		314	39049
125			316	21164	175		316	39464
126			318	21481	176		318	39881
127			320	21800	177		320	40300
128			322	22121	178		322	40721
129			324	22444	179		324	41144
130		Uniform Load = 2,000 pounds per foot	326	22769	180		326	41569
131			328	23096	181		328	41996
132			330	23425	182		330	42425
133			332	23756	183		332	42856
134			334	24089	184		334	43289
135			336	24424	185		336	43724
136			338	24761	186		338	44161
137			340	25100	187		340	44600
138			342	25441	188		342	45041
139			344	25784	189		344	45484
140		Uniform Load = 2,000 pounds per foot	346	26129	190		346	45929
141			348	26476	191		348	46376
142			350	26825	192		350	46825
143			352	27176	193		352	47276
144			354	27529	194		354	47729
145			356	27884	195		356	48184
146			358	28241	196		358	48641
147			360	28600	197		360	49100
148			362	28961	198		362	49561
149			364	29324	199		364	50024
150			366	29689	200		366	50489

COOPER'S E40. 200'-250'

COOPER'S E40. 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 Sum	Moment Sum	Length	Load	Load Sum	Moment Sum
300	Uniform Load = 2,000 pounds per foot	666	107089	350	Uniform Load = 2,000 pounds per foot	706	142809
301		668	107756	351		708	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 E50. 0'-50'

COOPER'S E50. 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 E50. 100'-150'

COOPER'S E50. 150'-200'

Length	Wheel	Load	Load Sum	Moment Sum	Length	Load	Load Sum	Moment Sum
100				17725.00	150		457.50	37111.25
101				17663.75	151		460.00	37270.00
102				18002.50	152		462.50	38031.25
103				18341.25	153		465.00	38495.00
104	w. 18	16.25	355.00	18680.00	154		467.50	38961.25
105				19035.00	155		470.00	39430.00
106				19390.00	156		472.50	39901.25
107				19745.00	157		475.00	40375.00
108				20100.00	158		477.50	40851.25
109			355.00	20455.00	159		480.00	41330.00
110			357.50	20811.25	160		482.50	41811.25
111			360.00	21170.00	161		485.00	42295.00
112			362.50	21531.25	162		487.50	42781.25
113			365.00	21895.00	163		490.00	43270.00
114			367.50	22261.25	164		492.50	43761.25
115			370.00	22630.00	165		495.00	44255.00
116			372.50	23001.25	166		497.50	44751.25
117			375.00	23375.00	167		500.00	45250.00
118			377.50	23751.25	168		502.50	45751.25
119			380.00	24130.00	169		505.00	46255.00
120			382.50	24511.25	170		507.50	46761.25
121			385.00	24895.00	171		510.00	47270.00
122			387.50	25281.25	172		512.50	47781.25
123			390.00	25670.00	173		515.00	48295.00
124			392.50	26061.25	174		517.50	48811.25
125			395.00	26455.00	175		520.00	49330.00
126			397.50	26851.25	176		522.50	49851.25
127			400.00	27250.00	177		525.00	50375.00
128			402.50	27651.25	178		527.50	50901.25
129			405.00	28055.00	179		530.00	51430.00
130			407.50	28461.25	180		532.50	51961.25
131			410.00	28870.00	181		535.00	52495.00
132			412.50	29281.25	182		537.50	53031.25
133			415.00	29695.00	183		540.00	53570.00
134			417.50	30111.25	184		542.50	54111.25
135			420.00	30530.00	185		545.00	54655.00
136			422.50	30951.25	186		547.50	55201.25
137			425.00	31375.00	187		550.00	55750.00
138			427.50	31801.25	188		552.50	56301.25
139			430.00	32230.00	189		555.00	56855.00
140			432.50	32661.25	190		557.50	57411.25
141			435.00	33095.00	191		560.00	57970.00
142			437.50	33531.25	192		562.50	58531.25
143			440.00	33970.00	193		565.00	59095.00
144			442.50	34411.00	194		567.50	59661.25
145			445.00	34855.00	195		570.00	60230.00
146			447.50	35301.25	196		572.50	60801.25
147			450.00	35750.00	197		575.00	61375.00
148			452.50	36201.25	198		577.50	61951.25
149			455.00	36655.00	199		580.00	62530.00
150			457.50	37111.25	200		582.50	63111.25

Uniform Load = 2,500 pounds per foot

Uniform Load = 2,500 pounds per foot

COOPER'S E50. 200'-250'				COOPER'S E50. 250'-300			
Length	Load	Load Sums	Moment Sums	Length	Load	Load Sums	Moment Sums
200	Uniform Load = 2,500 pounds per foot	582.50	63111.25	250	Uniform Load = 2,500 pounds per foot	707.50	95361.25
201		585.00	63695.00	251		710.00	96070.00
202		587.50	64281.25	252		712.50	96781.25
203		590.00	64870.00	253		715.00	97495.00
204		592.50	65461.25	254		717.50	98211.25
205		595.00	66055.00	255		720.00	98930.00
206		597.50	66651.25	256		722.50	99651.25
207		600.00	67250.00	257		725.00	100375.00
208		602.50	67851.25	258		727.50	101101.25
209		605.00	68455.00	259		730.00	101830.00
210		607.50	69061.25	260		732.50	102561.25
211		610.00	69670.00	261		735.00	103295.00
212		612.50	70281.25	262		737.50	104031.25
213		615.00	70895.00	263		740.00	104770.00
214		617.50	71511.25	264		742.50	105511.25
215		620.00	72130.00	265		745.00	106255.00
216		622.50	72751.25	266		747.50	107001.25
217		625.00	73375.00	267		750.00	107750.00
218		627.50	74001.25	268		752.50	108501.25
219		630.00	74630.00	269		755.00	109255.00
220		632.50	75261.25	270		757.50	110011.25
221		635.00	75895.00	271		760.00	110770.00
222		637.50	76531.25	272		762.50	111531.25
223		640.00	77170.00	273		765.00	112295.00
224		642.50	77811.25	274		767.50	113061.25
225		645.00	78455.00	275		770.00	113830.00
226		647.50	79101.25	276		772.50	114601.25
227		650.00	79750.00	277		775.00	115375.00
228		652.50	80401.25	278		777.50	116151.25
229		655.00	81055.00	279		780.00	116930.00
230		657.50	81711.25	280		782.50	117711.25
231		660.00	82370.00	281		785.00	118495.00
232		662.50	83031.25	282		787.50	119281.25
233		665.00	83695.00	283		790.00	120070.00
234		667.50	84361.25	284		792.50	120861.25
235		670.00	85030.00	285		795.00	121655.00
236		672.50	85701.25	286		797.50	122451.25
237		675.00	86375.00	287		800.00	123250.00
238		677.50	87051.25	288		802.50	124051.25
239		680.00	87730.00	289		805.00	124855.00
240		682.50	88411.25	290		807.50	125661.25
241		685.00	89095.00	291		810.00	126470.00
242		687.50	89781.25	292		812.50	127281.25
243		690.00	90470.00	293		815.00	128095.00
244		692.50	91161.25	294		817.50	128911.25
245		695.00	91855.00	295		820.00	129730.00
246		697.50	92551.25	296		822.50	130551.25
247		700.00	93250.00	297		825.00	131375.00
248		702.50	93951.25	298		827.50	132201.25
249		705.00	94655.00	299		830.00	133030.00
250		707.50	95361.25	300		832.50	133861.25

COOPER'S E50. 300'-350'

COOPER'S E50. 250'-300'

Length	Load	Load Stress	Moment Stress	Length	Load	Load Stress	Moment Stress
300		832.50	133861.25	350		957.50	179611.25
301		835.00	134695.00	351		960.00	179670.00
302		837.50	135531.25	352		962.50	180531.25
303		840.00	136370.00	353		965.00	181405.00
304		842.50	137211.25	354		967.50	182461.25
305		845.00	138055.00	355		970.00	183430.00
306		847.50	138901.25	356		972.50	184401.25
307		850.00	139750.00	357		975.00	185375.00
308		852.50	140601.25	358		977.50	186351.25
309		855.00	141455.00	359		980.00	187330.00
310		857.50	142311.25	360		982.50	188311.25
311		860.00	143170.00	361		985.00	189295.00
312		862.50	144031.25	362		987.50	190281.25
313		865.00	144895.00	363		990.00	191270.00
314		867.50	145761.25	364		992.50	192261.25
315		870.00	146630.00	365		995.00	193255.00
316		872.50	147501.25	366		997.50	194251.25
317		875.00	148375.00	367		1000.00	195250.00
318		877.50	149251.25	368		1002.50	196251.25
319		880.00	150130.00	369		1005.00	197255.00
320		882.50	151011.25	370		1007.50	198261.25
321		885.00	151895.00	371		1010.00	199270.00
322		887.50	152781.25	372		1012.50	200281.25
323		890.00	153670.00	373		1015.00	201295.00
324		892.50	154561.25	374		1017.50	202311.25
325		895.00	155455.00	375		1020.00	203330.00
326		897.50	156351.25	376		1022.50	204351.25
327		900.00	157250.00	377		1025.00	205375.00
328		902.50	158151.25	378		1027.50	206401.25
329		905.00	159055.00	379		1030.00	207430.00
330		907.50	159961.25	380		1032.50	208461.25
331		910.00	160870.00	381		1035.00	209495.00
332		912.50	161781.25	382		1037.50	210531.25
333		915.00	162695.00	383		1040.00	211570.00
334		917.50	163611.25	384		1042.50	212611.25
335		920.00	164530.00	385		1045.00	213655.00
336		922.50	165451.25	386		1047.50	214701.25
337		925.00	166375.00	387		1050.00	215750.00
338		927.50	167301.25	388		1052.50	216801.25
339		930.00	168230.00	389		1055.00	217855.00
340		932.50	169161.25	390		1057.50	218911.25
341		935.00	170095.00	391		1060.00	219970.00
342		937.50	171031.25	392		1062.50	221031.25
343		940.00	171970.00	393		1065.00	222095.00
344		942.50	172911.25	394		1067.50	223161.25
345		945.00	173855.00	395		1070.00	224230.00
346		947.50	174801.25	396		1072.50	225301.25
347		950.00	175750.00	397		1075.00	226375.00
348		952.50	176701.25	398		1077.50	227451.25
349		955.00	177655.00	399		1080.00	228530.00
350		957.50	178611.25	400		1082.50	229611.25

Uniform Load = 2,500 pounds per foot

Uniform Load = 2,500 pounds per foot

COOPER'S E60. 0'-50'

COOPER'S E60. 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

LIVE-LOAD STRESSES

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COOPER'S E60. 100'-150'

COOPER'S E60. 150'-200'

Length	Wheel	Load	Load Sum	Moment Sum	Length	Load	Load Sum	Moment Sum
100				20790.00	150		549.0	44533.50
101				21196.50	151		552.0	45084.00
102				21603.00	152		555.0	45637.50
103				22009.50	153		558.0	46194.00
104	w. 18	19.5	426.0	22416.00	154		561.0	46753.50
105				22842.00	155		564.0	47316.00
106				23268.00	156		567.0	47881.50
107				23694.00	157		570.0	48450.00
108				24120.00	158		573.0	49021.50
109			426.0	24546.00	159		576.0	49600.00
110			429.0	24973.50	160		579.0	50173.50
111			432.0	25404.00	161		582.0	50754.00
112			435.0	25837.50	162		585.0	51337.50
113			438.0	26274.00	163		588.0	51924.00
114			441.0	26713.50	164		591.0	52513.50
115			444.0	27156.00	165		594.0	53106.00
116			447.0	27601.50	166		597.0	53701.50
117			450.0	28050.00	167		600.0	54300.00
118			453.0	28501.50	168		603.0	54901.50
119			456.0	28956.00	169		606.0	55506.00
120			459.0	29413.50	170		609.0	56113.50
121			462.0	29874.00	171		612.0	56724.00
122			465.0	30337.50	172		615.0	57337.50
123			468.0	30804.00	173		618.0	57954.00
124			471.0	31273.50	174		621.0	58573.50
125			474.0	31746.00	175		624.0	59196.00
126			477.0	32221.50	176		627.0	59821.50
127			480.0	32700.00	177		630.0	60450.00
128			483.0	33181.50	178		633.0	61081.50
129			486.0	33666.00	179		636.0	61716.00
130			489.0	34153.50	180		639.0	62353.50
131			492.0	34644.00	181		642.0	62994.00
132			495.0	35137.50	182		645.0	63637.50
133			498.0	35634.00	183		648.0	64284.00
134			501.0	36133.50	184		651.0	64933.50
135			504.0	36636.00	185		654.0	65586.00
136			507.0	37141.50	186		657.0	66241.50
137			510.0	37650.00	187		660.0	66900.00
138			513.0	38161.50	188		663.0	67561.50
139			516.0	38676.00	189		666.0	68226.00
140			519.0	39193.50	190		669.0	68893.50
141			522.0	39714.00	191		672.0	69564.00
142			525.0	40237.50	192		675.0	70237.50
143			528.0	40764.00	193		678.0	70914.00
144			531.0	41293.50	194		681.0	71593.50
145			534.0	41826.00	195		684.0	72276.00
146			537.0	42361.50	196		687.0	72961.50
147			540.0	42900.00	197		690.0	73650.00
148			543.0	43441.50	198		693.0	74341.50
149			546.0	43986.00	199		696.0	75036.00
150			549.0	44533.50	200		699.0	75733.50

Uniform Load = 3,000 pounds per foot

Uniform Load = 3,000 pounds per foot

COOPER'S E60. 200'-250'

COOPER'S E60. 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 E60. 300'-450'

COOPER'S E60. 450'-600'

Length	Load	Load Sums	Moment Sums	Length	Load	Load Sums	Moment Sums
300		999.0	160033.50	350		1149.0	214333.50
301		1002.0	161634.00	351		1152.0	215484.00
302		1005.0	162637.50	352		1155.0	216637.50
303		1008.0	163644.00	353		1158.0	217794.00
304		1011.0	164653.50	354		1161.0	218953.50
305		1014.0	165666.00	355		1164.0	220116.00
306		1017.0	166681.50	356		1167.0	221281.50
307		1020.0	167700.00	357		1170.0	222450.00
308		1023.0	168721.50	358		1173.0	223621.50
309		1026.0	169746.00	359		1176.0	224796.00
310		1029.0	170773.50	360		1179.0	225973.50
311		1032.0	171804.00	361		1182.0	227154.00
312		1035.0	172837.50	362		1185.0	228337.50
313		1038.0	173874.00	363		1188.0	229524.00
314		1041.0	174913.50	364		1191.0	230713.50
315		1044.0	175956.00	365		1194.0	231906.00
316		1047.0	177001.50	366		1197.0	233101.50
317		1050.0	178050.00	367		1200.0	234300.00
318		1053.0	179101.50	368		1203.0	235501.50
319		1056.0	180156.00	369		1206.0	236706.00
320		1059.0	181213.50	370		1209.0	237913.50
321		1062.0	182274.00	371		1212.0	239124.00
322		1065.0	183337.50	372		1215.0	240337.50
323		1068.0	184404.00	373		1218.0	241554.00
324		1071.0	185473.50	374		1221.0	242773.50
325		1074.0	186546.00	375		1224.0	243996.00
326		1077.0	187621.50	376		1227.0	245221.50
327		1080.0	188700.00	377		1230.0	246450.00
328		1083.0	189781.50	378		1233.0	247681.50
329		1086.0	190866.00	379		1236.0	248916.00
330		1089.0	191953.50	380		1239.0	250153.50
331		1092.0	193044.00	381		1242.0	251394.00
332		1095.0	194137.50	382		1245.0	252637.50
333		1098.0	195234.00	383		1248.0	253884.00
334		1101.0	196333.50	384		1251.0	255133.50
335		1104.0	197436.00	385		1254.0	256386.00
336		1107.0	198541.50	386		1257.0	257641.50
337		1110.0	199650.00	387		1260.0	258900.00
338		1113.0	200761.50	388		1263.0	260161.50
339		1116.0	201876.00	389		1266.0	261426.00
340		1119.0	202993.50	390		1269.0	262693.50
341		1122.0	204114.00	391		1272.0	263964.00
342		1125.0	205237.50	392		1275.0	265237.50
343		1128.0	206364.00	393		1278.0	266514.00
344		1131.0	207493.50	394		1281.0	267793.50
345		1134.0	208626.00	395		1284.0	269076.00
346		1137.0	209761.50	396		1287.0	270361.50
347		1140.0	210900.00	397		1290.0	271650.00
348		1143.0	212041.50	398		1293.0	272941.50
349		1146.0	213186.00	399		1296.0	274236.00
350		1149.0	214333.50	400		1299.0	275533.50

COMMON STANDARD 0'-50'

COMMON STANDARD 50'-100'

Length	Wheel	Load	Load Sums	Moment Sums	Length	Wheel	Load	Load Sums	Moment Sums
0	w. 1	12.5	12.5	00.00	50				5120.00
1				12.50	51				5312.50
2				25.00	52				5505.00
3				37.50	53				5697.50
4				50.00	54				5890.00
5				62.50	55				6082.50
6				75.00	56	w. 10	12.5	205.0	6275.00
7				87.50	57				6480.00
8	w. 2	27.5	40.0	100.00	58				6685.00
9				140.00	59				6890.00
10				180.00	60				7095.00
11				220.00	61				7300.00
12				260.00	62				7505.00
13	w. 3	27.5	67.5	300.00	63				7710.00
14				367.50	64	w. 11	27.5	232.5	7915.00
15				435.00	65				8147.50
16				502.50	66				8380.00
17				570.00	67				8612.50
18	w. 4	27.5	95.0	637.50	68				8845.00
19				732.50	69	w. 12	27.5	260.0	9077.50
20				827.50	70				9337.50
21				922.50	71				9597.50
22				1017.50	72				9857.50
23	w. 5	27.5	122.5	1112.50	73				10117.50
24				1235.00	74	w. 13	27.5	287.5	10377.50
25				1357.50	75				10665.00
26				1480.00	76				10952.50
27				1602.50	77				11240.00
28				1725.00	78				11527.50
29				1847.50	79	w. 14	27.5	315.0	11815.00
30				1970.00	80				12130.00
31				2092.50	81				12445.00
32	w. 6	17.5	140.0	2215.00	82				12760.00
33				2355.00	83				13075.00
34				2495.00	84				13390.00
35				2635.00	85				13705.00
36				2775.00	86				14020.00
37	w. 7	17.5	157.5	2915.00	87				14335.00
38				3072.50	88	w. 15	17.5	332.5	14650.00
39				3230.00	89				14982.50
40				3387.50	90				15315.00
41				3545.00	91				15647.50
42				3702.50	92				15980.00
43	w. 8	17.5	175.0	3860.00	93	w. 16	17.5	350.0	16312.50
44				4035.00	94				16662.50
45				4210.00	95				17012.50
46				4385.00	96				17362.50
47				4560.00	97				17712.50
48	w. 9	17.5	192.5	4735.00	98				18062.50
49				4927.50	99	w. 17	17.5	367.5	18412.50
50				5120.00	100				18780.00

LIVE-LOAD STRESSES

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COMMON STANDARD 100'-150'

COMMON STANDARD 150'-200'

Length	Wheel	Load	Load Sum	Moment Sum	Length	Load	Load Sum	Moment Sum
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.				160		512.5	45061.25
111					161		515.0	45575.00
112					162		517.5	46091.25
113					163		520.0	46610.00
114					164		522.5	47131.25
115					165		525.0	47655.00
116					166		527.5	48181.25
117					167		530.0	48710.00
118					168		532.5	49241.25
119					169		535.0	49775.00
120	Uniform Load = 2,500 pounds per foot.				170		537.5	50311.25
121					171		540.0	50850.00
122					172		542.5	51391.25
123					173		545.0	51935.00
124					174		547.5	52481.25
125					175		550.0	53030.00
126					176		552.5	53581.25
127					177		555.0	54135.00
128					178		557.5	54691.25
129					179		560.0	55250.00
130	Uniform Load = 2,500 pounds per foot.				180		562.5	55811.25
131					181		565.0	56375.00
132					182		567.5	56941.25
133					183		570.0	57510.00
134					184		572.5	58081.25
135					185		575.0	58655.00
136					186		577.5	59231.25
137					187		580.0	59810.00
138					188		582.5	60391.25
139					189		585.0	60975.00
140	Uniform Load = 2,500 pounds per foot.				190		587.5	61561.25
141					191		590.0	62150.00
142					192		592.5	62741.25
143					193		595.0	63335.00
144					194		597.5	63931.25
145					195		600.0	64530.00
146					196		602.5	65131.25
147					197		605.0	65735.00
148					198		607.5	66341.25
149					199		610.0	66950.00
150					200		612.5	67561.25

COMMON STANDARD 200'-250'

COMMON STANDARD 250'-300'

Length	Load	Load Sums	Moment Sums	Length	Load	Load Sums	Moment Sums
200	Uniform Load = 2,500 pounds per foot	612.5	67561.25	250	Uniform Load = 2,500 pounds per foot	737.5	101311.25
201		615.0	68175.00	251		740.0	102050.00
202		617.5	68791.25	252		742.5	102791.25
203		620.0	69410.00	253		745.0	103535.00
204		622.5	70031.25	254		747.5	104281.25
205		625.0	70655.00	255		750.0	105030.00
206		627.5	71281.25	256		752.5	105781.25
207		630.0	71910.00	257		755.0	106535.00
208		632.5	72541.25	258		757.5	107291.25
209		635.0	73175.00	259		760.0	108050.00
210		637.5	73811.25	260		762.5	108811.25
211		640.0	74450.00	261		765.0	109575.00
212		642.5	75091.25	262		767.5	110341.25
213		645.0	75735.00	263		770.0	111110.00
214		647.5	76381.25	264		772.5	111881.25
215		650.0	77030.00	265		775.0	112655.00
216		652.5	77681.25	266		777.5	113431.25
217		655.0	78335.00	267		780.0	114210.00
218		657.5	78991.25	268		782.5	114991.25
219		660.0	79650.00	269		785.0	115775.00
220		662.5	80311.25	270		787.5	116561.25
221		665.0	80975.00	271		790.0	117350.00
222		667.5	81641.25	272		792.5	118141.25
223		670.0	82310.00	273		795.0	118935.00
224		672.5	82981.25	274		797.5	119731.25
225		675.0	83655.00	275		800.0	120530.00
226		677.5	84331.25	276		802.5	121331.25
227		680.0	85010.00	277		805.0	122135.00
228		682.5	85691.25	278		807.5	122941.25
229		685.0	86375.00	279		810.0	123750.00
230		687.5	87061.25	280		812.5	124561.25
231		690.0	87750.00	281		815.0	125375.00
232		692.5	88441.25	282		817.5	126191.25
233		695.0	89135.00	283		820.0	127010.00
234		697.5	89831.25	284		822.5	127831.25
235		700.0	90530.00	285		825.0	128655.00
236		702.5	91231.25	286		827.5	129481.25
237		705.0	91935.00	287		830.0	130310.00
238		707.5	92641.25	288		832.5	131141.25
239		710.0	93350.00	289		835.0	131975.00
240		712.5	94061.25	290		837.5	132811.25
241		715.0	94775.00	291		840.0	133650.00
242		717.5	95491.25	292		842.5	134491.25
243		720.0	96210.00	293		845.0	135335.00
244		722.5	96931.25	294		847.5	136181.25
245		725.0	97655.00	295		850.0	137030.00
246		727.5	98381.25	296		852.5	137881.25
247		730.0	99110.00	297		855.0	138735.00
248		732.5	99841.25	298		857.5	139591.25
249		735.0	100575.00	299		860.0	140450.00
250		737.5	101311.25	300		862.5	141311.25

COMMON STANDARD 300'-350'

COMMON STANDARD 350'-400'

Length	Load	Load Sum	Moment Sum	Length	Load	Load Sum	Moment Sum
300		862.5	141311.25	350		987.50	187561.25
301		865.0	142175.00	351		990.00	188350.00
302		867.5	143041.25	352		992.50	189141.25
303		870.0	143910.00	353		995.00	190035.00
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

Uniform Load = 2,500 pounds per foot

Uniform Load = 2,500 pounds per foot

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'

LACKAWANNA 150'-200'

Length	Wheel	Load	Load Sum	Moment Sum	Length	Load	Load Sum	Moment Sum
100	w. 18	14	334.00	17170.000	150		437.50	30250.500
101		..		17504.000	151		439.75	30680.125
102		..		17838.000	152		442.00	31130.000
103		..		18172.000	153		444.25	31573.125
104			334.00	18506.000	154		446.50	32018.500
105			336.25	18841.125	155		448.75	32466.125
106			338.50	19178.500	156		451.00	32916.000
107			340.75	19518.125	157		453.25	33368.125
108			343.00	19860.000	158		455.50	33822.500
109			345.25	20204.125	159		457.75	34279.125
110			347.50	20550.500	160		460.00	34738.000
111			349.75	20899.125	161		462.25	35199.125
112			352.00	21250.000	162		464.50	35662.500
113			354.25	21603.125	163		466.75	36128.125
114			356.50	21958.500	164		469.00	36596.000
115			358.75	22316.125	165		471.25	37066.125
116			361.00	22676.000	166		473.50	37538.500
117			363.25	23038.125	167		475.75	38013.125
118			365.50	23402.500	168		478.00	38490.000
119			367.75	23769.125	169		480.25	38969.125
120			370.00	24138.000	170		482.50	39450.500
121			372.25	24509.125	171		484.75	39934.125
122			374.50	24882.500	172		487.00	40420.000
123			376.75	25258.125	173		489.25	40908.125
124			379.00	25636.000	174		491.50	41398.500
125			381.25	26016.125	175		493.75	41891.125
126			383.50	26398.500	176		496.00	42386.000
127			385.75	26783.125	177		498.25	42883.125
128			388.00	27170.000	178		500.50	43382.500
129			390.25	27559.125	179		502.75	43884.125
130			392.50	27950.500	180		505.00	44388.000
131			394.75	28344.125	181		507.25	44894.125
132			397.00	28740.000	182		509.50	45402.500
133			399.25	29138.125	183		511.75	45913.125
134			401.50	29538.500	184		514.00	46426.000
135			403.75	29941.125	185		516.25	46941.125
136			406.00	30346.000	186		518.50	47458.500
137			408.25	30753.125	187		520.75	47978.125
138			410.50	31162.500	188		523.00	48500.000
139			412.75	31574.125	189		525.25	49024.125
140			415.00	31988.000	190		527.50	49550.500
141			417.25	32404.125	191		529.75	50079.125
142			419.50	32882.500	192		532.00	50610.000
143			421.75	33243.125	193		534.25	51143.125
144			424.00	33666.000	194		536.50	51678.500
145			426.25	34091.125	195		538.75	52216.125
146			428.50	34518.500	196		541.00	52756.000
147			430.75	34948.125	197		543.25	53298.125
148			433.00	35380.000	198		545.50	53842.500
149			435.25	35814.125	199		547.75	54389.125
150			437.50	36250.500	200		550.00	54938.000

Uniform Load = 2,250 pounds per foot

Uniform Load = 2,250 pounds per foot

LACKAWANNA 200'-250'

LACKAWANNA 250'-300'

Length	Load	Load Sums	Moment Sums	Length	Load	Load Sums	Moment Sums
Uniform Load = 2,250 pounds per foot				Uniform Load = 2,250 pounds per foot			
200		550.00	60938.000	250		662.50	91250.500
201		552.25	61489.125	251		664.75	91914.125
202		554.50	62042.500	252		667.00	92580.000
203		556.75	62598.125	253		669.25	93248.125
204		559.00	63156.000	254		671.50	93918.500
205		561.25	63716.125	255		673.75	94591.125
206		563.50	64278.500	256		676.00	95266.000
207		565.75	64843.125	257		678.25	95943.125
208		568.00	65410.000	258		680.50	96622.500
209		570.25	65979.125	259		682.75	97304.125
210		572.50	66550.500	260		685.00	97988.000
211		574.75	67124.125	261		687.25	98674.125
212		577.00	67700.000	262		689.50	99362.500
213		579.25	68278.125	263		691.75	100053.125
214		581.50	68858.500	264		694.00	100746.000
215		583.75	69441.125	265		696.25	101441.125
216		586.00	70026.000	266		698.50	102138.500
217		588.25	70613.125	267		700.75	102838.125
218		590.50	71202.500	268		703.00	103540.000
219		592.75	71794.125	269		705.25	104244.125
220		595.00	72388.000	270		707.50	105950.500
221		597.25	72984.125	271		709.75	106659.125
222		599.50	73582.500	272		712.00	106370.000
223		601.75	74183.125	273		714.25	107083.125
224		604.00	74786.000	274		716.50	107798.500
225		606.25	75391.125	275		718.75	108516.125
226		608.50	75998.500	276		721.00	109236.000
227		610.75	76608.125	277		723.25	109958.125
228		613.00	77220.000	278		725.50	110682.500
229		615.25	77834.125	279		727.75	111409.125
230		617.50	78450.500	280		730.00	112138.000
231		619.75	79069.125	281		732.25	112869.125
232		622.00	79690.000	282		734.50	113602.500
233		624.25	80313.125	283		736.75	114338.125
234		626.50	80938.500	284		739.00	115076.000
235		628.75	81566.125	285		741.25	115816.125
236		631.00	82196.000	286		743.50	116558.500
237		633.25	82828.125	287		745.75	117303.125
238		635.50	83462.500	288		748.00	118050.000
239		637.75	84099.125	289		750.25	118799.125
240		640.00	84738.000	290		752.50	119550.500
241		642.25	85379.125	291		754.75	120304.125
242		644.50	86022.500	292		757.00	121060.000
243		646.75	86668.125	293		759.25	121818.125
244		649.00	87316.000	294		761.50	122578.500
245		651.25	87966.125	295		763.75	123341.125
246		653.50	88618.500	296		766.00	124106.000
247		655.75	89273.125	297		768.25	124873.125
248		658.00	89930.000	298		770.50	125642.500
249		660.25	90589.125	299		772.75	126414.125
250		662.50	91250.500	300		775.00	127188.000

LACKAWANNA 300'-350'

LACKAWANNA 350'-400'

Length	Load	Load Sum	Moment Sum	Length	Load	Load Sum	Moment Sum
300		775.00	127188.000	350		887.50	168750.500
301		777.25	127904.125	351		889.75	169639.125
302		779.50	128742.500	352		892.00	170530.000
303		781.75	129523.125	353		894.25	171423.125
304		784.00	130306.000	354		896.50	172318.500
305		786.25	131091.125	355		898.75	173216.125
306		788.50	131878.500	356		901.00	174116.000
307		790.75	132668.125	357		903.25	175018.125
308		793.00	133460.000	358		905.50	175922.500
309		795.25	134254.125	359		907.75	176829.125
310		797.50	135050.500	360		910.00	177738.000
311		799.75	135849.125	361		912.25	178649.125
312		802.00	136650.000	362		914.50	179562.500
313		804.25	137453.125	363		916.75	180478.125
314		806.50	138258.500	364		919.00	181396.000
315		808.75	139066.125	365		921.25	182316.125
316		811.00	139876.000	366		923.50	183238.500
317		813.25	140688.125	367		925.75	184163.125
318		815.50	141502.500	368		928.00	185090.000
319		817.75	142319.125	369		930.25	186019.125
320		820.00	143138.000	370		932.50	186950.500
321		822.25	143959.125	371		934.75	187884.125
322		824.50	144782.500	372		937.00	188820.000
323		826.75	145608.125	373		939.25	189758.125
324		829.00	146436.000	374		941.50	190698.500
325		831.25	147266.125	375		943.75	191641.125
326		833.50	148098.500	376		946.00	192586.000
327		835.75	148933.125	377		948.25	193533.125
328		838.00	149770.000	378		950.50	194482.500
329		840.25	150609.125	379		952.75	195434.125
330		842.50	151450.500	380		955.00	196388.000
331		844.75	152294.125	381		957.25	197344.125
332		847.00	153140.000	382		959.50	198302.500
333		849.25	153988.125	383		961.75	199263.125
334		851.50	154838.500	384		964.00	200226.000
335		853.75	155691.125	385		966.25	201191.125
336		856.00	156546.000	386		968.50	202158.500
337		858.25	157403.125	387		970.75	203128.125
338		860.50	158262.500	388		973.00	204100.000
339		862.75	159124.125	389		975.25	205074.125
340		865.00	159988.000	390		977.50	206050.500
341		867.25	160854.125	391		979.75	207029.125
342		869.50	161722.500	392		982.00	208010.000
343		871.75	162593.125	393		984.25	208993.125
344		874.00	163466.000	394		986.50	209978.500
345		876.25	164341.125	395		988.75	210966.125
346		878.50	165218.500	396		991.00	211956.000
347		880.75	166098.125	397		993.25	212948.125
348		883.00	166980.000	398		995.50	213942.500
349		885.25	167864.125	399		997.75	214939.125
350		887.50	168750.500	400		1000.00	215938.000

TABLE 3
POSITION OF COOPER'S LOADINGS FOR MAXIMUM STRESS
Shorter Segment l_1

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	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	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	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	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	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	14	14	14	13	13	11	12	12	12	13	13	13	..	..	..	..	..
95	2	3	3	4	4	5	13	13	13	13	13	13	13	13	12	12	12	13	13	..	..	..	..	..	..
90	2	3	3	4	4	5	13	13	13	13	13	13	13	13	12	12	12	13	..	..	..	..	..	..	..
85	2	3	3	4	4	5	13	13	12	13	13	12	13	13	12	12	12	..	..	..	..	..	..	..	..
80	2	3	3	4	4	13	13	13	12	12	12	12	12	12	12	12	..	..	..	..	..	..	..	..	..
75	2	3	3	4	4	13	13	12	12	12	12	12	12	12	12	12	..	..	..	..	..	..	..	..	..
70	2	3	3	4	4	13	13	12	12	12	12	11	11	11	..	..	..	..	..	..	..	..	..	..	..
65	2	3	3	4	4	12	12	12	12	12	11	11	11	..	..	..	..	..	..	..	..	..	..	..	..
60	11	3	3	4	4	5	13	12	11	11	11	11	..	..	..	..	..	..	..	..	..	..	..	..	..
55	11	12	12	12	4	12	13	12	12	13	11	..	..	..	..	..	..	..	..	..	..	..	..	..	..
50	11	12	12	12	12	12	13	13	13	13	12	..	..	..	..	..	..	..	..	..	..	..	..	..	..
45	2	3	12	12	12	12	13	13	13	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
40	2	3	3	3	12	12	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	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 = " " " " 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_1 at left end	Left end
23 " 27	Right to left	w_2 at right end	Right end
27 " 46	Right to left	w_2 at left end	Left end
46 " 62	Right to left	w_1 at left end	Left end
62 " 400	Right to left	w_1 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
8	5-6	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	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
9	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
	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
10	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
	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	B16					A16				
	Max. Moment	Max. Shears			Max. Pier React.	Max. Moment	Max. Shears			Max. Pier React.
		End	$\frac{1}{4}$ Pt.	Cent.			End	$\frac{1}{4}$ 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	E40					E50				
	Max. Moment	Max. Shears			Max. Pier React.	Max. Moment	Max. Shears			Max. Pier React.
		End	¼ Pt.	Cent.			End	¼ Pt.	Cent.	
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	E40					E50				
	Max. Moment	Max. Shears			Max. Pier React.	Max. Moment	Max. Shears			Max. Pier React.
		End	1/4 Pl.	Cent.			End	1/4 Pl.	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

NOTE.—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
	3	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	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	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	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	3	5617	5993	6390	6790	7204	7620	8054	8496	8959	9415	9892
	4	6187	6610	7040	7485	7966	8460	8980	9490	10010	10530	11065

TABLE 8.—Continued

MAXIMUM MOMENTS FOR TRUSS BRIDGES—COOPER'S E50 FOR ONE RAIL

Moments Given in Thousands of Foot-Pounds

Panel Points $\overset{0}{|} \overset{1}{|} \overset{2}{|} \overset{3}{|} \overset{4}{|} \overset{5}{|} \overset{6}{|} \overset{7}{|} \overset{8}{|} \overset{9}{|}$

Panes 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"	25' 0"
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	3745	3883	
5	1	2600	2731	2864	3001	3138	3279	3418	3562	3704	3852	3999	
	2	3685	3943	4144	4347	4555	4767	4978	5188	5415	5646	5881	
6	1	3210	3362	3516	3678	3840	4008	4175	4349	4522	4700	4878	
	2	4885	5256	5591	5950	6250	6601	6956	7311	7677	8047	8421	
	3	5487	5746	6028	6321	6617	6921	7223	7533	7850	8165	8491	
7	1	3778	3955	4130	4317	4505	4702	4897	5100	5308	5512	5721	
	2	6025	6326	6613	6914	7215	7520	7845	8173	8508	8845	9182	
	3	7140	7646	7990	8347	8710	9079	9448	9826	10207	10590	10971	
8	1	4320	4525	4727	4939	5150	5373	5592	5829	6061	6290	6520	
	2	7125	7458	7805	8162	8520	8890	9260	9640	10020	10407	10800	
	3	8780	9234	9530	10070	10516	10995	11475	11976	12472	12961	13450	
	4	94.0	9943	10396	10862	11317	11805	12288	12790	13297	13795	14290	
9	1	4850	5179	5508	5845	6180	6520	6862	7204	7549	7894	8240	
	2	8195	8578	8970	9378	9790	10216	10649	11082	11525	11968	12410	
	3	10372	10880	11375	11900	12425	12978	13555	14118	14705	15290	15871	
	4	11605	12172	12735	13310	13880	14472	15068	15684	16209	16800	17360	

Panes 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"	30' 6"
3	1	2135	2215	2289	2370	2451	2534	2616	2700	2792	2888	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	5763	
5	1	4150	4301	4456	4611	4770	4929	5092	5253	5422	5589	5760	
	2	6093	6371	6552	6783	7017	7250	7492	7734	7984	8232	8482	
6	1	5061	5245	5433	5622	5816	6010	6208	6408	6612	6817	7026	
	2	7794	8058	8352	8654	8960	9268	9580	9897	10219	10547	10880	
	3	8821	9153	9490	9828	10170	10514	10862	11208	11565	11923	12284	
7	1	5936	6151	6373	6595	6823	7051	7286	7521	7762	8003	8250	
	2	9530	9875	10236	10600	10980	11357	11742	12123	12500	12891	13280	
	3	11444	11870	12312	12762	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	15720	
	3	14010	14528	15063	15605	16163	16718	17285	17852	18431	19018	19600	
	4	14820	15340	15875	16413	16965	17514	18073	18633	19190	19759	20306	
9	1	7622	7900	8188	8477	8774	9070	9376	9686	9996	10310	10620	
	2	12925	13400	13890	14380	14885	15400	15900	16400	16905	17417	17930	
	3	16528	17145	17778	18414	19070	19730	20405	21080	21770	22461	23160	
	4	18205	18850	19515	20180	20870	21557	22260	22955	23679	24405	25170	

TABLE 8.—*Continued*

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										
		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 E50 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											
		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	40.6	42.1	43.5	44.8	46.4	47.9	49.1	50.4	51.6	52.9	54.3	55.9
	2	7.3	8.0	8.8	9.5	10.0	11.0	11.8	12.5	13.2	14.7	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	77.4
	2	23.5	25.4	27.4	28.6	30.0	31.3	32.4	33.4	34.4	35.4	36.7	37.7
5	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
	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
6	2	38.8	41.0	43.0	44.9	46.7	48.7	50.3	51.9	53.5	55.3	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	27.4	28.7
7	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	67.9	70.1	72.1	74.2	76.3	78.3
8	3	30.2	32.5	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
9	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	98.0
10	3	43.4	45.6	48.0	50.4	52.4	54.8	56.9	58.3	59.6	62.0	64.3	66.9
	4	24.1	26.0	27.6	29.0	30.5	32.1	33.4	34.7	36.1	37.4	38.6	39.9
11	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
	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
12	2	78.2	81.7	85.2	89.1	92.5	96.0	99.5	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	79.3	82.2	84.9
13	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	32.9	33.9
14	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
	1	115.2	122.3	129.2	135.6	141.9	148.4	154.5	160.8	166.4	172.0	177.4	183.0
15	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	94.9	98.2	101.9	106.4
16	4	48.2	51.1	53.8	56.5	58.5	60.8	63.1	65.1	67.4	69.3	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
17	6	16.6	17.5	18.1	20.3	21.5	22.7	23.9	25.0	26.2	27.3	28.3	29.3

Panels in Truss	Panel	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	57.4	59.7	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	157.3	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	194.6	198.4	203.1	207.8	212.3	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	60.8	61.4	63.1	64.8	66.7	68.5	70.4	72.2	74.0	75.9	
	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	20.9	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 E50 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	
	3	100.7	103.0	105.6	108.2	110.7	113.2	115.5	117.7	120.0	122.2	124.4	
4	1	50.3	51.3	52.2	53.1	54.0	54.9	55.8	56.8	57.4	58.2	59.4	
	2	14.7	15.0	15.3	15.6	15.9	16.2	16.5	16.7	17.0	17.2	17.5	
	3	133.5	136.6	139.8	142.9	146.0	149.0	152.0	154.9	157.8	160.5	163.3	
5	1	77.4	79.1	80.9	82.6	84.4	86.1	88.0	89.9	91.7	93.5	95.1	
	2	38.1	38.8	39.6	40.3	40.9	41.6	42.3	42.9	43.7	44.3	45.0	
	3	164.6	168.1	171.7	175.2	178.8	182.3	185.8	189.2	192.6	195.9	199.2	
6	1	108.6	111.0	113.6	116.0	118.5	120.8	123.2	125.4	127.9	130.1	132.4	
	2	62.1	63.5	65.1	66.6	68.2	69.6	71.3	72.9	74.5	75.9	77.4	
	3	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	
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	
	1	221.7	226.3	230.8	235.2	239.8	244.3	248.9	253.4	258.0	262.5	267.1	
9	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	
6	5	45.2	46.1	47.1	48.0	49.0	49.4	51.0	52.1	53.1	54.1	55.3	
	1	21.9	22.4	22.9	23.4	23.9	24.4	24.9	25.3	25.7	26.0	26.5	
	2	248.8	253.9	259.0	264.0	269.2	274.2	279.4	284.5	289.7	294.8	299.9	
3	3	195.4	199.5	203.5	207.5	211.5	215.5	219.4	223.3	227.2	231.0	234.9	
	4	147.4	150.6	153.8	156.9	160.0	163.0	166.0	169.0	172.0	175.0	177.9	
	5	104.9	107.3	109.7	112.0	114.3	116.6	118.9	121.1	123.4	125.5	127.8	
5	6	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	
	3	126.5	128.7	130.9	133.1	135.2	137.3	139.3	141.5	143.6	145.8	147.9	
4	1	59.7	60.5	61.3	62.1	62.9	63.8	64.6	65.6	66.5	67.4	68.3	
	2	17.8	18.1	18.4	18.6	18.9	19.1	19.3	19.6	19.8	20.1	20.3	
	3	166.0	168.8	171.4	174.1	176.7	179.4	181.9	184.5	187.0	189.6	192.0	
5	1	96.6	98.3	100.1	101.9	103.6	105.4	107.1	108.9	110.6	112.3	114.0	
	2	45.5	46.3	46.9	47.7	48.3	49.0	49.6	50.5	51.3	52.1	52.8	
	3	202.5	205.8	209.0	212.2	215.4	218.6	221.8	224.9	228.0	231.1	234.2	
6	1	134.5	136.8	139.0	141.3	143.5	145.8	148.0	150.3	152.4	154.6	156.7	
	2	78.6	80.2	81.5	83.0	84.3	85.7	87.0	88.4	89.6	91.1	92.4	
	3	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	
8	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	32.3	32.6	33.0	33.5	33.8	34.3	34.6	35.1	35.6	
	1	271.5	276.0	280.4	284.9	289.2	293.6	297.9	302.3	306.5	310.8	315.0	
9	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	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	
	1	304.9	310.0	315.0	320.1	325.0	330.0	334.9	339.9	344.7	349.7	354.5	
6	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	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 E50 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"	36' 0"
3	1	101.1	102.6	104.6	106.6	108.1	109.6	111.5	113.4	114.8	116.2	117.8	119.4
	2	30.4	30.8	31.2	31.5	31.8	32.2	32.5	32.8	33.1	33.4	33.7	34.0
4	1	149.9	152.0	154.0	156.1	158.0	160.0	161.9	163.8	165.8	167.9	169.9	172.0
	2	69.1	70.0	71.7	73.3	74.4	75.4	76.4	77.4	78.4	79.4	80.4	81.4
	3	20.6	20.9	21.1	21.3	21.6	22.0	22.2	22.5	22.7	23.0	23.3	23.6
5	1	194.6	197.1	199.8	202.4	205.0	207.5	210.1	212.6	215.1	217.6	220.2	222.7
	2	115.6	117.3	118.9	120.4	122.0	123.5	125.0	126.5	128.0	129.5	131.0	132.5
	3	53.6	54.3	55.1	55.9	56.7	57.4	58.3	59.1	60.0	60.9	61.7	62.6
6	1	237.3	240.3	243.5	246.6	249.8	252.9	256.0	259.1	262.2	265.4	268.5	271.6
	2	158.8	160.9	163.0	165.1	167.2	169.3	171.4	173.4	175.4	177.4	179.4	181.4
	3	93.7	95.0	96.3	97.5	98.8	100.0	101.3	102.5	103.8	105.1	106.4	107.6
	4	43.0	43.6	44.4	45.1	45.8	46.4	47.2	47.9	48.6	49.3	50.0	50.7
7	1	278.7	282.3	286.0	289.6	293.4	297.1	300.9	304.7	308.4	312.0	315.7	319.4
	2	200.6	203.3	205.9	208.5	211.2	213.8	216.4	218.9	221.5	224.0	226.6	229.1
	3	133.6	135.3	137.1	138.9	140.7	142.5	144.3	146.0	147.9	149.8	151.7	153.5
	4	79.0	80.1	81.3	82.4	83.5	84.5	85.6	86.6	87.7	88.7	89.8	90.8
	5	36.1	36.5	37.0	37.5	38.0	38.5	39.2	39.9	40.5	41.0	41.6	42.1
8	1	319.3	323.5	327.8	332.0	337.0	341.9	345.6	349.3	353.2	357.0	360.9	364.7
	2	241.4	244.6	247.8	251.0	254.2	257.4	260.6	263.8	266.9	270.0	273.2	276.4
	3	172.8	175.4	177.8	180.1	182.5	184.8	187.1	189.4	191.7	193.9	196.2	198.5
	4	115.7	117.3	118.7	120.3	121.9	123.4	124.9	126.3	127.7	129.1	130.5	131.9
	5	67.9	68.9	69.9	70.9	71.9	72.9	73.9	74.8	75.7	76.6	77.5	78.4
	6	31.2	31.5	32.0	32.5	32.9	33.3	33.8	34.3	34.7	35.1	35.5	35.9
9	1	359.4	364.2	369.1	373.9	378.7	383.5	388.5	393.5	398.4	403.3	408.2	413.1
	2	281.6	285.4	289.2	293.0	296.8	300.5	304.3	308.0	311.8	315.5	319.2	322.9
	3	212.4	215.3	218.2	221.0	223.9	226.8	229.6	232.5	235.3	238.1	240.9	243.8
	4	152.7	154.8	156.8	158.8	160.7	162.6	164.6	166.6	168.6	170.5	172.4	174.4
	5	101.8	103.1	104.5	105.9	107.3	108.6	110.0	111.4	112.7	114.0	115.4	116.7
	6	59.4	60.3	61.2	62.0	62.9	63.8	64.7	65.5	66.3	67.1	67.9	68.8

TABLE 10

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

	SHORTER SEGMENT l_1											
	5	10	15	20	25	30	35	40	45	50	55	60
Longer Segment l_2												
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	31591	33033	34455
225	16639	17862	19123	20351	21569	22757	23908	25078	27315	29502	31691	33862
200	14939	16036	17172	18269	19360	20418	21440	22482	24465	26400	28331	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	22900	24446
150	11487	12354	13194	14058	14887	15681	16442	17231	18706	20151	21569	22966
140	10790	11608	12395	13206	13980	14722	15430	16169	17542	18870	20206	21520
130	10088	10857	11594	12349	13069	13756	14413	15101	16372	17600	18834	20055
120	9380	10100	10786	11486	12073	12787	13421	14026	15197	16325	17414	18485
110	8666	9338	9972	10616	11226	11812	11392	12946	14014	15044	16034	17005
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. $M = l_1 l_2 + 3800 \frac{l_1}{L}$

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

	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. $M = 1.25 l_1 l_2 + 4750 \frac{l_2}{L}$

TABLE 11.—Continued

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

	65	70	75	80	85	90	95	100	110	120	130	140
250	22909	24594	26327	28026	29707	31355	32955	34575	37000	40739	43791	46819
225	20709	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	23600	24835	26996	29083	31204	33299
160	15231	16371	17523	18633	19736	20795	21812	22861	24832	26745	28602	30518
150	14359	15443	16492	17573	18609	19601	20553	21539	23382	25180	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. $M = 1.25 l_1 l_2 + 4750 \frac{l_1}{L}$

TABLE 12

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

	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	5855	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. $M = 1.5 \frac{l_1 l_2}{L} + 5700 \frac{l_2}{L}$

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 h_1

	65	70	75	80	85	90	95	100	110	120	130	140
250	27491	29513	31592	33631	35648	37626	39546	41490	43228	48887	52549	56185
225	24959	26792	28685	30527	32353	34135	35862	37616	40973	44254	47537	50792
200	22409	24054	25758	27403	29040	30626	32160	33722	36007	39600	42497	45281
175	19836	21307	22811	24257	25700	27025	28428	29802	32395	34918	37444	39947
160	18277	19645	21028	22360	23683	24964	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 h_1 and h_2 each > 142 ft. $M = 1.5 h_1 h_2 + 5700 \frac{h_1^2}{L}$

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											
		0	5	10	15	20	25	30	35	40	45	50	55
Longer Segment l_2	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. $R = L + \frac{3800}{l_1}$

TABLE 14

MAXIMUM PIER REACTIONS BETWEEN EQUAL AND UNEQUAL SPANS, COOPER'S
E50 LOADING

Values in Thousands of Pounds per Rail

		SHORTER SEGMENT l_1											
		0	5	10	15	20	25	30	35	40	45	50	55
Longer Segment l_2	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
E50 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	356	365	374	385
130...	286	291	299	304	312	317	323	328	334	343	352	362	371
120...	272	278	285	291	299	303	310	316	321	331	340	350	359
110...	259	265	273	279	287	292	298	304	309	319	329	339	349
100...	246	253	260	267	274	280	286	291	296	307	317	327	337
95...	240	247	254	260	267	274	279	286	291	302	312	322	332
90...	235	242	248	254	261	268	273	280	286	297	307	317	327
85...	229	236	242	248	255	261	267	273	279	290	300	310	320
80...	223	230	235	242	249	255	261	267	273	284	294	304	314
75...	216	222	229	235	242	248	254	260	266	277	287	297	307
70...	208	214	222	228	234	240	246	252	258	269	279	289	299
65...	200	206	214	220	226	232	238	244	250	261	271	281	291
60...	191	197	205	211	217	223	229	235	241	252	262	272	282

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

TABLE 15
MAXIMUM PIER REACTIONS BETWEEN EQUAL AND UNEQUAL SPANS, COOPER'S
E60 LOADING

Values in Thousands of Pounds per Rail

		SHORTER SEGMENT l_1											
		0	5	10	15	20	25	30	35	40	45	50	55
Longer Segment l_2	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. $R = 1.5 L + \frac{5760}{l_1}$

TABLE 15.—Continued

MAXIMUM PIER REACTIONS BETWEEN EQUAL AND UNEQUAL SPANS, COOPER'S
E60 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	401	409	415	426	438	449	462
130...	343	349	359	365	374	380	388	394	401	412	422	434	446
120...	326	334	342	349	359	364	372	379	385	397	408	420	432
110...	311	318	328	335	344	350	358	365	371	383	394	406	418
100...	295	304	312	320	329	336	343	349	356	368	379	391	403
95...	288	296	305	312	320	329	335	343	349	361	372	384	396
90...	282	290	298	305	313	322	328	335	342	354	365	377	389
85...	275	283	290	298	306	313	321	328	335	347	358	370	382
80...	268	276	282	290	299	306	314	321	328	340	351	363	375
75...	259	266	275	282	291	300	307	315	322	334	345	357	369
70...	250	257	266	274	283	292	300	308	315	327	338	350	362
65...	240	247	256	264	273	282	290	298	305	317	328	340	352
60...	229	236	245	253	262	271	279	287	294	306	317	329	341

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

TABLE 16

EQUIVALENT UNIFORM LOADS FOR COOPER'S E40 LOADING

Values in Pounds per Lineal Foot per Rail

		SHORTER SEGMENT l_1											
		0	5	10	15	20	25	30	35	40	45	50	55
Longer Segment l_2	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 E40 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	2280	2280	2270	2270	2260	2260	2250	2240	2230	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	2190	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	2230	2200
130	2400	2390	2390	2380	2380	2370	2350	2340	2330	2290	2260	2230	2200
120	2420	2410	2410	2400	2400	2370	2370	2350	2340	2300	2270	2230	2200
110	2440	2420	2420	2420	2420	2400	2390	2380	2350	2320	2280	2230	2200
100	2460	2460	2450	2440	2440	2420	2410	2390	2380	2340	2300	2230	2200
95	2500	2480	2460	2460	2450	2440	2420	2400	2390	2340	2300	2230	2200
90	2510	2500	2480	2460	2460	2450	2430	2400	2390	2340	2300	2230	2200
85	2530	2510	2500	2490	2470	2460	2430	2400	2390	2340	2300	2230	2200
80	2550	2540	2520	2500	2490	2460	2430	2400	2390	2340	2300	2230	2200
75	2560	2540	2530	2510	2490	2460	2430	2400	2390	2340	2300	2230	2200
70	2560	2540	2530	2510	2490	2460	2430	2400	2390	2340	2300	2230	2200
65	2560	2540	2530	2510	2490	2460	2430	2400	2390	2340	2300	2230	2200
60	2550	2540	2530	2510	2490	2460	2430	2400	2390	2340	2300	2230	2200

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

TABLE 17
EQUIVALENT UNIFORM LOADS FOR COOPER'S E50 LOADING

Values in Pounds per Lineal Foot per Rail

		SHORTER SEGMENT l_1											
		0	5	10	15	20	25	30	35	40	45	50	55
Longer Segment l_2	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 E50 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	2730	2700	2680
225	2860	2850	2840	2840	2830	2820	2810	2800	2780	2770	2730	2710	2690
200	2890	2870	2860	2860	2850	2840	2830	2810	2790	2750	2730	2700	
175	2920	2900	2900	2900	2890	2880	2860	2850	2840	2800	2760	2750	2730
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	2765
120	3020	3005	3005	2995	2995	2980	2960	2940	2920	2880	2835	2795	2765
110	3045	3030	3030	3020	3015	3000	2985	2965	2940	2905	2855	2815	2775
100	3080	3065	3060	3050	3045	3030	3010	2985	2965	2925	2875	2835	2795
95	3115	3095	3075	3065	3060	3050	3020	3001	2980	2935	2885	2845	2805
90	3140	3120	3100	3080	3075	3060	3035	3015	2995	2945	2895	2855	2815
85	3160	3140	3120	3105	3090	3070	3045	3025	3005	2950	2900	2860	2820
80	3185	3165	3145	3125	3110	3090	3065	3045	3025	2965	2915	2875	2835
75	3200	3180	3155	3140	3120	3100	3075	3055	3035	2970	2920	2880	2840
70	3200	3180	3160	3140	3120	3100	3075	3055	3035	2965	2915	2875	2835
65	3200	3180	3160	3140	3120	3100	3075	3055	3035	2965	2915	2875	2835
60	3190	3170	3150	3130	3110	3090	3065	3045	3025	2955	2905	2865	2825

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

PLEASE RETURN TO
DEPT. of APPLIED MECHANICS.

TABLE 18
EQUIVALENT UNIFORM LOADS FOR COOPER'S E60 LOADING

Values in Pounds per Lineal Foot per Rail

		SHORTER SEGMENT l_1											
		0	5	10	15	20	25	30	35	40	45	50	55
Longer Segment l_2	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 E60 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	3290	3240	3230
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	3260	3200
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. $q = \left(3.0 + \frac{11400}{l_1 L}\right) 1000$

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

[illegible]

TABLE 19.—Continued

INFLUENCE-LINE ORDINATES FOR M FOR GIRDER BRIDGES WITHOUT FLOOR-BEAMSValues 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	77.8	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	79.5	83.0
175.....	47.2	50.0	52.4	54.9	57.1	59.3	61.7	63.7	67.1	71.4	75.7	79.1
160.....	46.1	48.5	51.0	53.2	55.6	57.9	60.1	62.4	66.8	71.1	75.4	79.6
150.....	45.2	47.6	50.0	52.1	54.3	56.2	58.1	60.3	64.6	68.9	73.2	77.5
140.....	44.4	46.7	49.0	51.0	52.9	54.6	56.5	58.1	62.4	66.7	71.0	75.0
130.....	43.3	45.5	47.6	49.5	51.3	53.0	54.6	56.5	60.2	64.5	68.8	72.8
120.....	42.2	44.3	46.3	48.1	49.8	51.5	53.2	54.6	57.9	62.2	66.5	70.5
110.....	40.8	42.7	44.6	46.3	48.1	49.8	51.5	52.4	55.0	59.3	63.6	67.5
100.....	39.4	41.2	42.9	44.4	46.1	47.4	48.8	50.0	52.4	56.7	61.0	65.0
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
RECIPROCAL OF INFLUENCE-LINE ORDINATES FOR M FOR GIRDER BRIDGES
WITHOUT FLOOR-BEAMS

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

SHORTER SEGMENT l_1

[illegible]

TABLE 20.—Continued

RECIPROCAL OF INFLUENCE-LINE ORDINATES FOR M FOR GIRDER BRIDGES
WITHOUT FLOOR-BEAMS

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

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	0148
120	0237	0226	0216	0208	0201	0194	0188	0183	0174	0167	0160	0154
110	0245	0234	0224	0216	0208	0202	0196	0191	0182	0174	0167	0160
100	0254	0243	0233	0225	0217	0211	0205	0200	0191	0182	0174	0167
95	0259	0248	0238	0230	0223	0216	0211	0205	0196	0187	0178	0170
90	0265	0254	0244	0236	0229	0222	0216	0211	0202	0193	0184	0175
85	0272	0261	0251	0243	0235	0228	0222	0216	0207	0198	0189	0180
80	0279	0268	0258	0250	0242	0234	0228	0222	0213	0204	0195	0186
75	0287	0276	0266	0258	0250	0242	0235	0229	0220	0211	0202	0193
70	0296	0286	0276	0268	0260	0252	0245	0238	0229	0220	0211	0202
65	0307	0296	0286	0278	0270	0262	0255	0248	0239	0230	0221	0212

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 4

[illegible]

TABLE 21.—Continued

BENDING MOMENTS IN BEAMS DUE TO UNIFORM LOAD OF 1 POUND PER
LINEAL FOOT

Values in Foot-pounds

Values equal $\frac{ll_1}{2} =$ 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	12250
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	9100
120	3900	4200	4500	4800	5100	5400	5700	6000	6600	7200	7800	8400
110	3575	3850	4125	4400	4675	4950	5225	5500	6050	6600	7150	7700
100	3250	3500	3750	4000	4250	4500	4750	5000	5500	6000	6500	7000
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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