AISC Orthotropic Plate Design Manual

W. A. MILEK, JR.

THE STRUCTURAL system with which we are concerned is orthogonal anisotropic plate (popularly termed "orthotropic"). This means orthogonal—at right angles—and anisotropic—different properties. Thus, we are talking about a steel plate that has different physical properties in mutually perpendicular directions. Can this be right? For all practical purposes, steel has the same modulus of elasticity, the same elastic limit, the same Poisson's ratio, the same ductility, and the same ultimate strength in all directions. Considering these facts, there cannot be such a thing as orthogonal anisotropic steel plate. There cannot be because steel is an isotropic material.

On the other hand, an assemblage of isotropic steel elements consisting of a steel plate stiffened and supported on a system of ribs, beams and girders may be thought of as having anisotropic properties. In other words, anisotropy is due not to different elastic properties of the material but to the physical dimensions and arrangement of the components of the assemblage. When we use the term "orthotropic steel deck" we are talking about bridges whose steel decks are *idealized* as orthotropic plates—all parts working together as a unit rather than individual pieces designed separately on the basis of their individual functions.

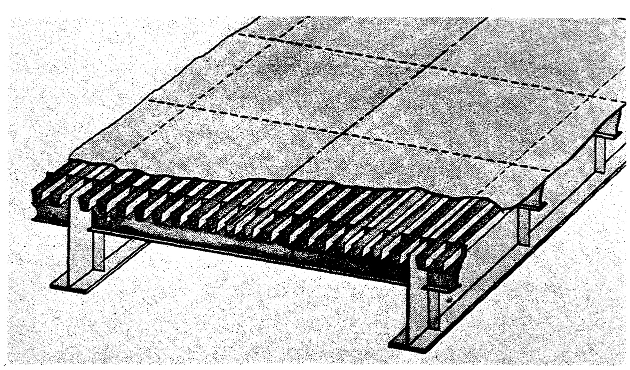

Figure 1

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Diagrammed in Fig. 1 is a section of a typical steel deck bridge. It consists of a wearing surface (not shown) supported by a deck plate welded to stiffening ribs, floor beams and main girders. The deck plate performs multiple functions. While distributing the wheel load laterally to the stiffeners it acts as the top flange of the stiffeners. It also acts as the top flange of the floor beams and girders. It is the deck plate that ties all elements together; thus a complicated system of shear, bending and axial stresses which vary continuously from point to point throughout the deck is involved.

At this stage it may appear that this concept is too complex for everyday practice. Considering the infinite number of combinations of bending, shear and axial forces that would exist in the expanse of a major bridge deck, too much design time would be required. This would be true if an analytical approach from scratch was required in the solution of a practical design. As with many problems which a designer handles more frequently, it is not now necessary to start from scratch. Much of the theoretical work—assembled from European studies and experience—has been applied to the specific case of bridge decks under AASHO loading. It has been digested, organized and presented in a useable manner. The AISC orthotropic manual[1] contains not only the theoretical background, but also design procedure recommendations and charts which eliminate much of the numerical work required and greatly simplify the remainder. Taken together, the theoretical developments give the designer an understanding of the proper use of charts and conversely, the charts give the designer a feel for the theory.

The primary value of the manual lies in the charts and the ease with which a steel deck can be proportioned through their use. A designer need not concern himself deeply with the theoretical development. The significance of several of the charts may not be apparent until after some study. Also the similarity of several charts may give rise to a feeling of uncertainty and a reticence to proceed.

1. Design Manual for Orthotropic Steel Plate Deck Bridges, *American Institute of Steel Construction, New York, 1963.*

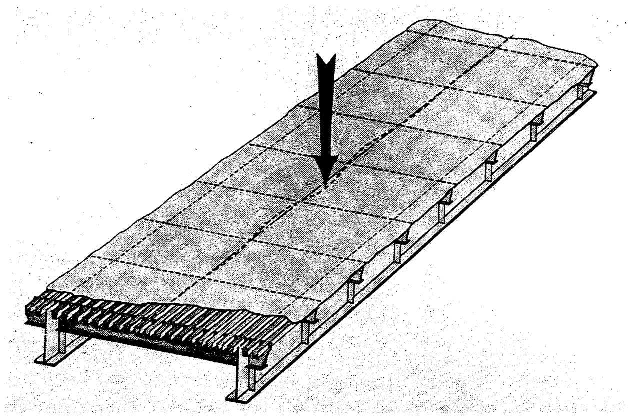


Figure 2

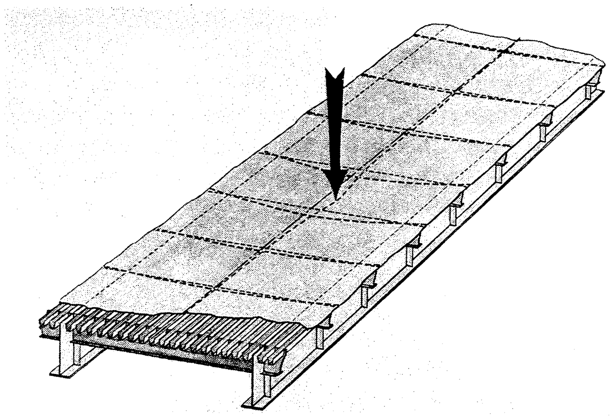


Figure 3

A few comments, which may serve as a sort of a road map, may be helpful.

Shown in Fig. 2 is a section of a hypothetical steel plate bridge deck. A concentrated load applied to the deck over a rib at mid-span between two floor beams, which for the moment are assumed to be infinitely rigid, will produce deflections along the centerline of the loaded rib approximately as shown by the center longitudinal dotted line. The moments induced would be similar to those produced by a conventional continuous beam on rigid supports. They would not be identical since the deck would distribute the load laterally as well as longitudinally, but it will be recognized that high positive moments would be induced under the load and high negative moments would be induced over the supports. In successive spans the moments and deflections would be carried over due to the support moments.

Of course the floor beams in a bridge deck are not infinitely rigid; therefore, if a load is applied to the deck, the reactions from the deck and ribs will cause floor beam deflections. In virtually every case, the floor beams will act as simple beams due to the fact that usually the main girders will be torsionally flexible. In some cases where box girders are employed or where more than two main girders are involved, this would not be true. The important thing to notice in Fig. 3 is that when a load is applied to the deck, the floor beams also deflect, as is to be expected. The floor beams nearest the point of application of the load deflect most, but those remote from the point of application are also affected. It might be said that the floor beam tends to "run away" from the reaction of the directly loaded rib. In so doing, the floor beam causes the positive moment at the center of the rib span to be increased and the negative moment in the deck over the floor beam to be decreased. Further, since the "running-away-from-the-deck" reaction causes a reduction of the reaction that would exist in an infinitely rigid system, the moment at the center of the floor beam is also reduced. The deflections throughout the length of the floor beam cause deflections throughout the width of the deck itself; thus a concentrated load at a single point of the deck calls upon a broad expanse of the deck for its support.

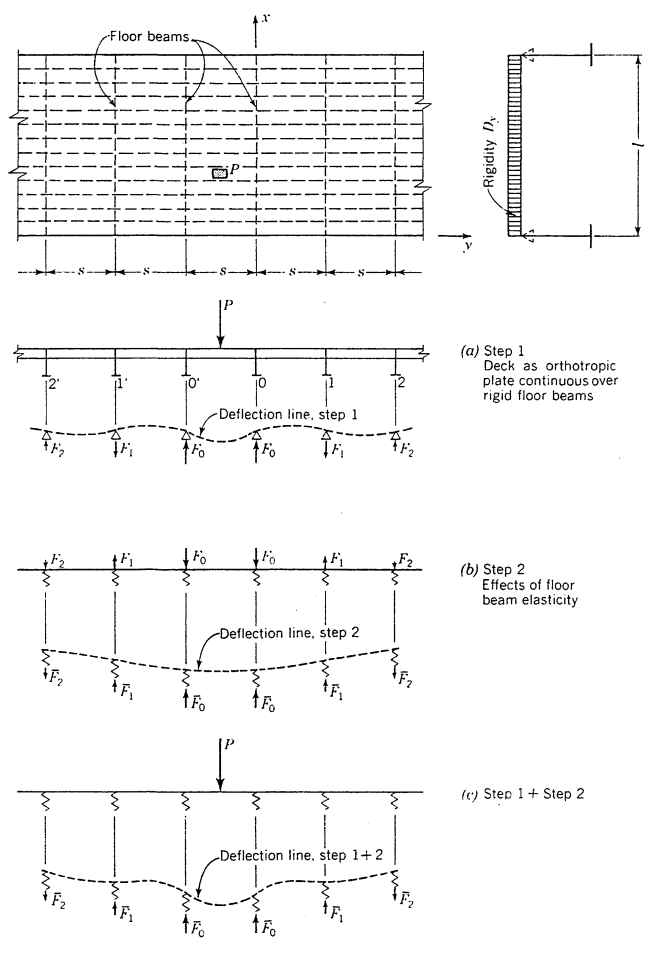


Fig. 4. The two-step computation procedure, Pelikan-Esslinger design method

GENERAL APPROACH TO A SOLUTION

The curves in Fig. 4 indicate the general approach to a solution of the problem. Moments may be calculated for the deck due to a concentrated load, assuming rigid support. Next the effect of flexibility of the support may be calculated and added to the rigid support moments. The summation of the effects of the critical combination of loads will be the information required for the design.

Influence lines for the moment at mid-span, at the support and the reaction at the support are shown in Fig. 5. γ is a function of the relative stiffness of the deck to the support beams. Thus for the case of a stiff deck on flexible floor beams (large values of γ) the increase in moment will be large. The reverse conditions would be represented by small values of γ. The parameter γ is important to the use of the charts and will be discussed later.

One more general diagram may be helpful before the use of charts in the solution of a problem is demonstrated. Fig. 6 shows the manner in which a deck plate on closed ribs deforms under the action of a concentrated load. As a directly loaded rib deflects, it not only works as a beam longitudinally—as shown in the previous figures—but also through shear and torsion there is a lateral distribution of the load. Thus the redundance of the complete system is of a high order.

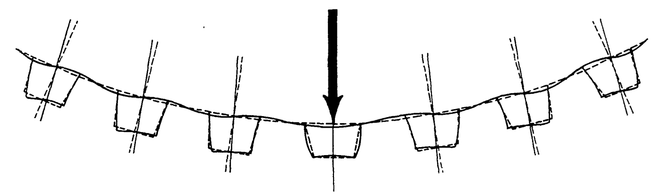


Figure 6

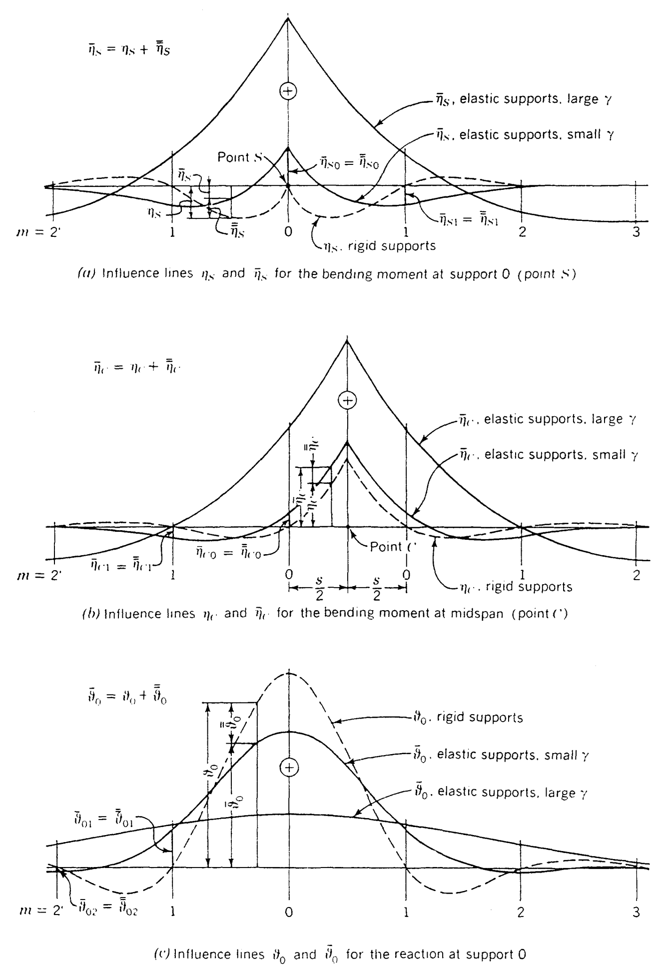


Fig. 5. Typical influence lines for continuous beams on elastic and on rigid supports

The problem is not as complicated as it may seem. Only a few areas of the deck need be investigated, such as the locations where the effects of local loading, action of the deck as the top flange of the floor beams, and action of the deck as the top flange of the main girders combine to produce maximum stress. These may be readily identified by inspection.

As a case in point, consider the hypothetical structure shown in Fig. 7. Point **A** at the mid-point of a rib span near the mid-point of a floor beam span would be critical and typical of numerous points on the bridge. Local bending stresses due to the directly applied load would be maximum since point **A** is at the mid-span between two floor beams. Since the point is near mid-span of the floor beam, the floor beam deflections would have their maximum effect in increasing the mid-span moments of the deck. Also since the point is near the mid-span of the floor beam, the stresses in the deck plate acting as the top flange of the floor beam would be maximum. Point **B** over a floor beam and at a point where stress in the main girders would be maximum would also be critical. If the maximum combined stresses are provided for at several such critical locations, the remainder of the deck will be conservatively stressed.

Proceeding with this same hypothetical structure, the use of the charts will be demonstrated. The typical cross section is as shown in Fig. 8. Note that the deck is stiffened with closed ribs. Floor beams are spaced 15 ft on center and span 50 ft between main girders. The main girders are single web girders; thus they would provide no end restraint to the floor beams. However, part of a traffic lane and a sidewalk are cantilevered beyond the main girders; therefore some dead load end restraint can be counted upon. Notice finally that the floor beam is divided horizontally. The bottom half, which is of an I-shape, would be erected in advance of the top half and would serve as the erection support for the deck panels.

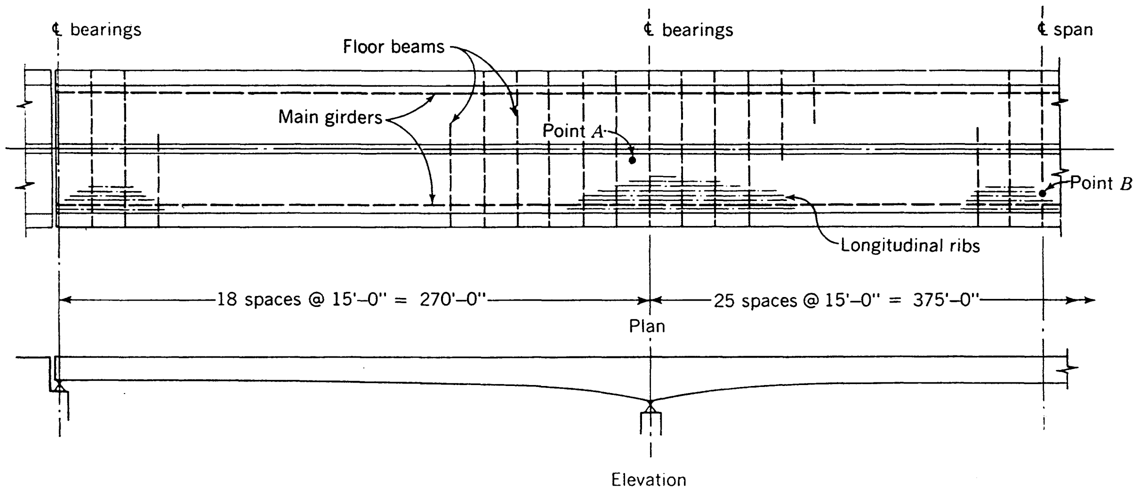


Fig. 7. Deck with closed ribs—general layout

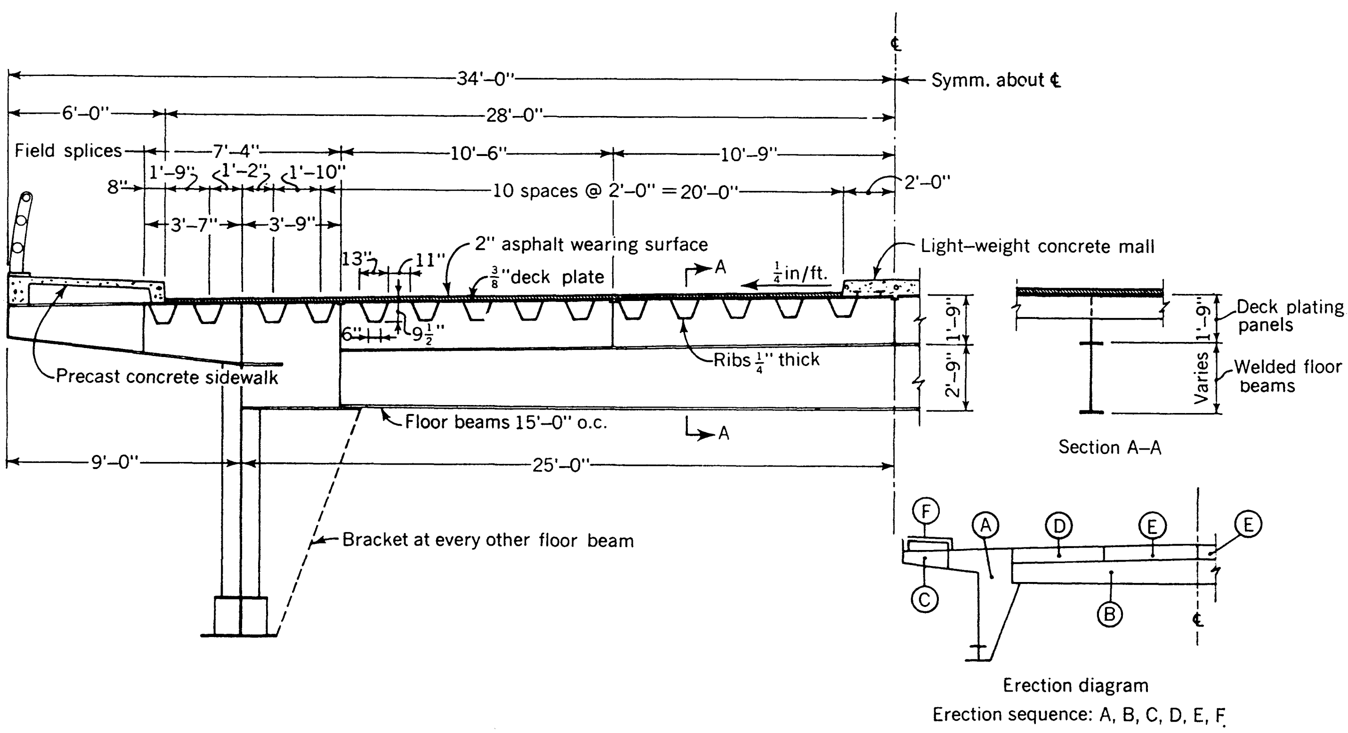


Fig. 8. Deck with closed ribs—typical cross section

DETERMINATION OF SECTION PROPERTIES

A schematic diagram, showing the steps required for the determination of section properties in a steel plate deck, is shown below.

As in conventional structures, start with the general layout. Choose a rib spacing based upon personal experience, or the experience of others. Many examples of bridges constructed in Europe are described in the first chapter of the orthotropic manual. A review of these examples will help the designer to make an initial selection of rib spacing.

Next, the plate thickness is determined. Due to the extremely redundant nature of the deck plate action in supporting local loads, stress calculations are largely meaningless at this stage. Tests have indicated that the ultimate strength of an orthotropic plate supporting concentrated loads is fifteen to twenty times that indicated by a theoretical stress analysis. However, the deflections of the deck plate are meaningful and important. They provide a simple means for deciding upon the thickness of the deck. Considering only AASHO loads and accepting a deflection of 1/300 of the plate span, a simple expression for plate thickness will result ($t_p = 0.007\ a\sqrt[3]{p}$). In this expression a is the plate span, p is unit pressure of the wheel (59 psi for 12 kip wheel). It is recommended that the 16 kip wheel load specified in the AASHO Specification to provide for occasional overloads not be used in view of the inherent high reserve capacity of the deck plate.

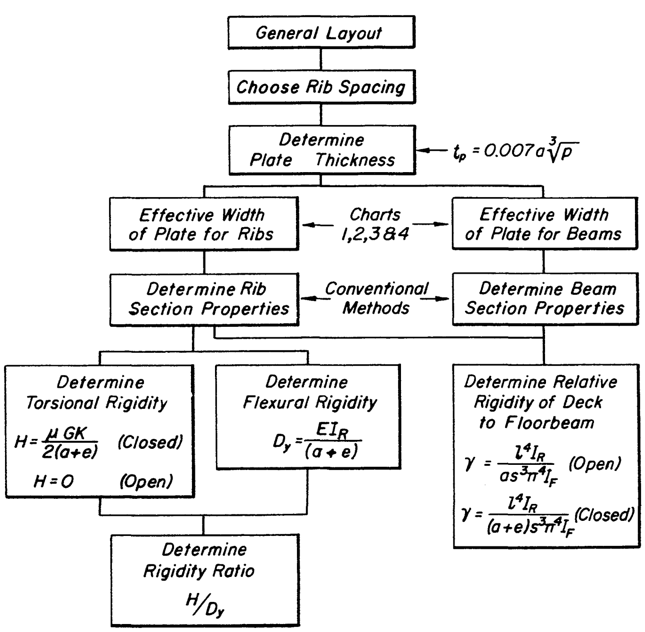
Entering the chart in Fig. 9 (Chart 1) with a given ratio of the *actual* width of plate to span length (plotted as abscissa), and reading the ordinates at the intercept with the plotted curve, the ratios of the *effective* width of plate to span length may be read. The chart may be used for determining the effective width of plate with either the ribs or the floor beams. The dotted line on the left is applicable to the deck plate between the ribs. The dotted line in the middle is applicable to the plate spanning between the two sides of the rib, and the dotted line on the right is applicable to the plate as the top flange of the floor beams. Charts 2, 3, and 4 in the manual provide information which is useful for handling special conditions of this same problem.

The effective width of deck plate acting as the top flange of the main girders is not treated in any charts or formulas. For design purposes, it may be assumed that the entire cross-sectional area of the plate acts as the top flange of the girders, provided the girder spacing is smaller than one-third the girder span.

With the thickness of the plate known the section properties are determined by the conventional formulas for the geometric properties of cross sections.

The next step is to determine several special properties and ratios which can be applied to this type construction. Considering first the deck plate and the ribs, determine the torsional rigidity, H, by the formula $H = \mu GK/[2(a + e)]$. In this expression, μ is a reduction coefficient depending upon the flexibility of the deck plate and the span of the ribs. It depends upon the geometry and span of the ribs and deck; thus it may be calculated by substituting appropriate numbers in formulas which are presented in Chapter 4 of the manual. The formulas contain many terms, but are not difficult since only arithmetical operations are involved. Also, in the formula

DETERMINATION OF REQUIRED SECTION PROPERTIES



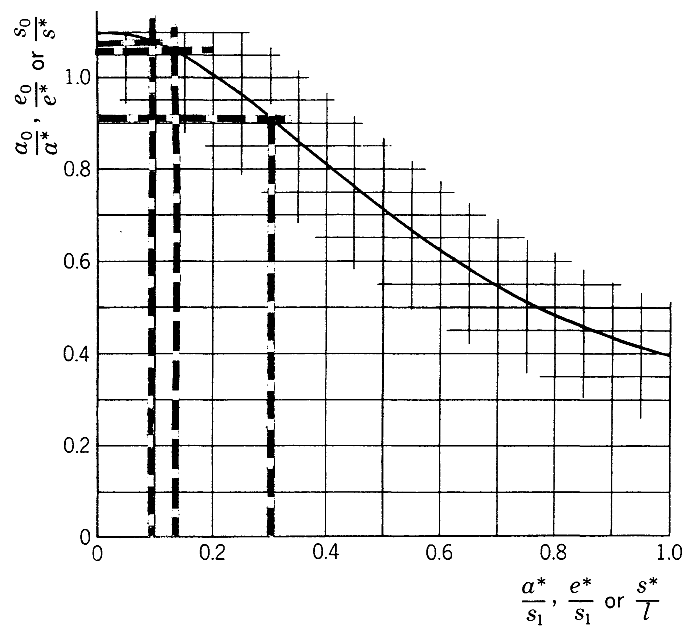


Chart 1. Effective width of deck acting with one rib or floor beam

Figure 9

for H, the factor K is a section property expressing torsional resistance and is equal to $4A^2/(ut_r + a/t_p)$. For open ribs, $H = 0$. The flexural rigidity of the reinforced deck plate in the longitudinal direction is calculated by $D_y = EI_R/(a + e)$. Once determined, these two properties are combined as the deck plate rigidity ratio, H/D_y, to be used later.

The relative rigidity coefficient of the stiffened deck plate to the floor beams, γ, must also be determined for future use. For closed ribs, $\gamma = l^4 I_R/(a + e)s^3\pi^4 I_F$. The relative rigidity of deck plate to rigidity of floor beam is the controlling factor in the amount of moment increase or relief when floor beam flexibility is taken into account.

It was pointed out earlier that the approximate method upon which the charts of the manual are based contemplates calculating first the moments in the ribs and floor beams on the assumption of rigid floor beams. The resulting moments are then modified to take account of the effect of floor beam flexibility. Continuing with the outline of the procedure, consider the moments in the deck plate on rigid supports.

MOMENTS IN RIBS ON RIGID SUPPORTS

Dead load moments in a rib (including the effective width of deck plate calculated earlier) may be determined by the usual formulas for moments in a uniformly loaded beam on rigid supports: M at supports equals $wl^2/12$ and M at mid-span equals $wl^2/24$.

Live load moments due to wheel loads as specified by AASHO Specifications are not as simple to determine for the closed rib sections. This is due partly to lateral distribution of the load to adjacent ribs and partly to the varying effect of placement and spacing of the loads in adjacent spans. The expression used for live load moments for a unit width of plate is:

$$M = Q_o s \sum_{n=1}^{\infty} \frac{Q_{nx}}{Q_o} \frac{\eta_n}{s} \qquad \text{Eq. (4.35)*}$$

The moment for a single rib including deck plate is

$$M_R = M\,(a + e)$$

Solutions to the equations for moment in a unit width of deck plate for various loadings are presented directly in Charts 9 through 13 of the manual. For example, using the rib span 15 ft and $H/D_y = 0.51$ calculated as previously outlined, the moments in a unit width of plate may be read directly for loading condition **a** from Fig. 10 (Chart 9), as 13 k-in./in. For loading condition $\mathbf{a_1}$ the moment will be 13.5 k-in./in. from Fig. 11 (Chart 11). For loading condition **e** the moment over the support will be −18.6 k-in./in., Fig. 12 (Chart 13).

* *Equation number as given in Reference 1*

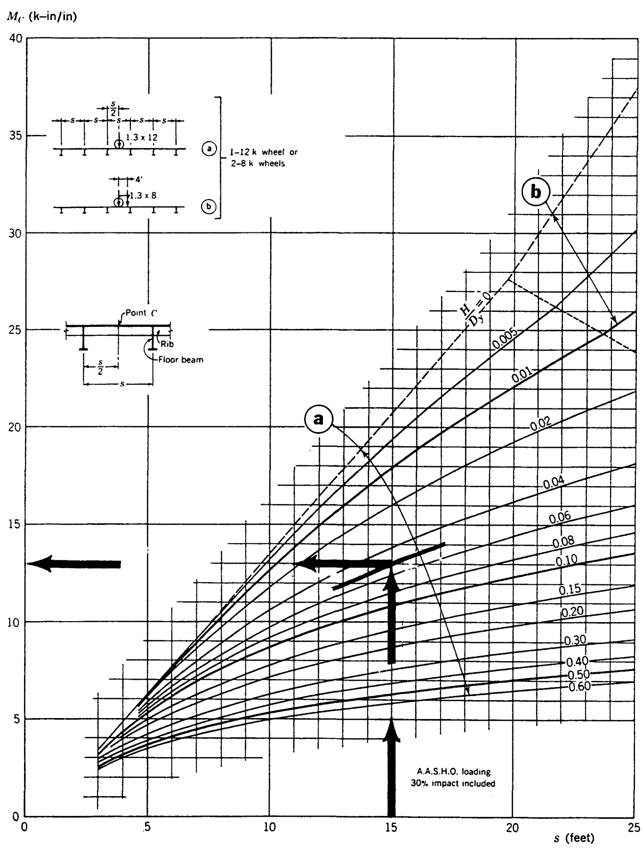


Chart 9. Maximum moment at midspan of the deck with closed ribs on rigid supports, loading a and b

Figure 10

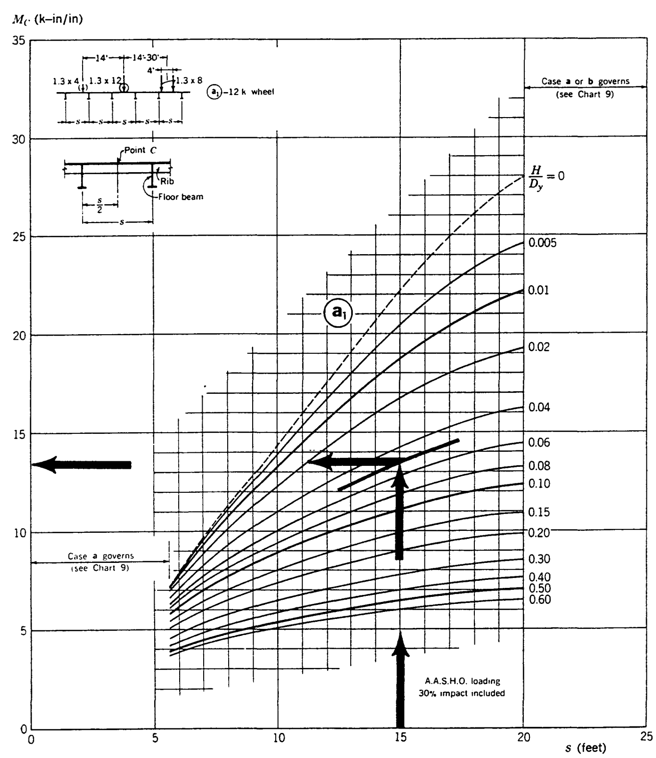


Chart 11. Maximum moment at midspan of the deck with closed ribs on rigid supports, loading a_1

Figure 11

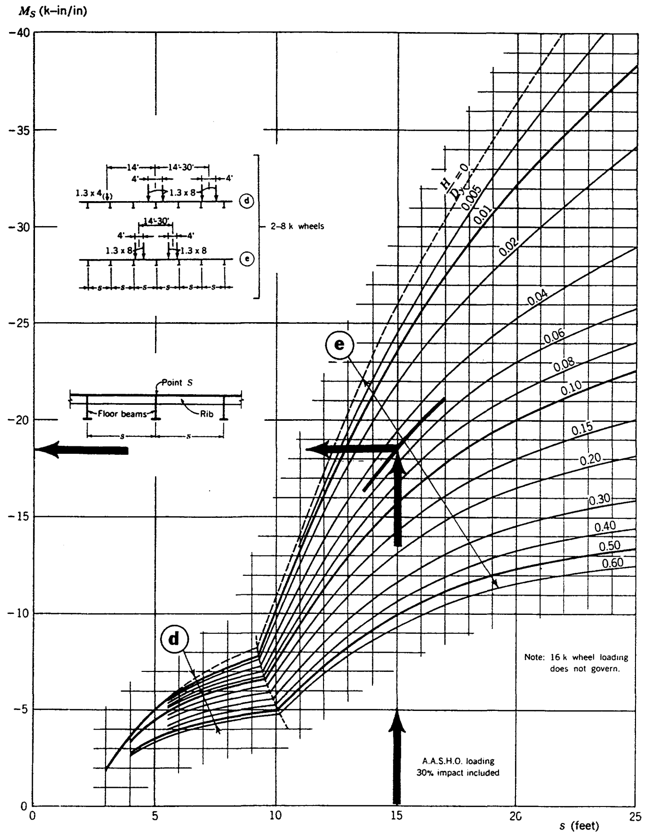


Chart 13. Maximum moment at support of the deck with closed ribs on rigid supports

Figure 12

EFFECT OF FLOOR BEAM FLEXIBILITY ON RIB MOMENTS

The moments thus determined multiplied by the width of one rib give the live load moments per rib for the stiffened deck on rigid supports.

The effect of floor beam flexibility may next be calculated by means of the equation

$$\Delta M_R = Q_o s(a + e) \frac{Q_{1x}}{Q_o} \sum \frac{F_m}{P} \frac{\bar{\eta}_{im}}{s} \qquad \text{Eq. (5.9a)*}$$

This expression is more involved and determination of a direct answer from a single chart is not possible. However, values for the several factors may be readily determined.

In this example, for loading condition **a**, Q_o is the unit load per unit of length under the load and is equal to 12 kips times the impact factor 1.3, divided by 22, the loaded width under a standard 12 kip wheel. The factor s is the span, 180 in. The factor $(a + e)$ is the rib spacing, equal to 24 in. The factor Q_{1x}/Q_o expresses the first sinusoidal component for a Fourier series. In this case Fig. 13 (Chart 28) gives data from which the value 0.126 for 50 ft span and 22 in. loaded width may be read. Fig. 14 (Chart 20) gives values for $\sum(F_m/P)(\bar{\eta}_{im}/s)$, which for the case in point is 0.0526.

* *Equation number as given in Reference 1.*

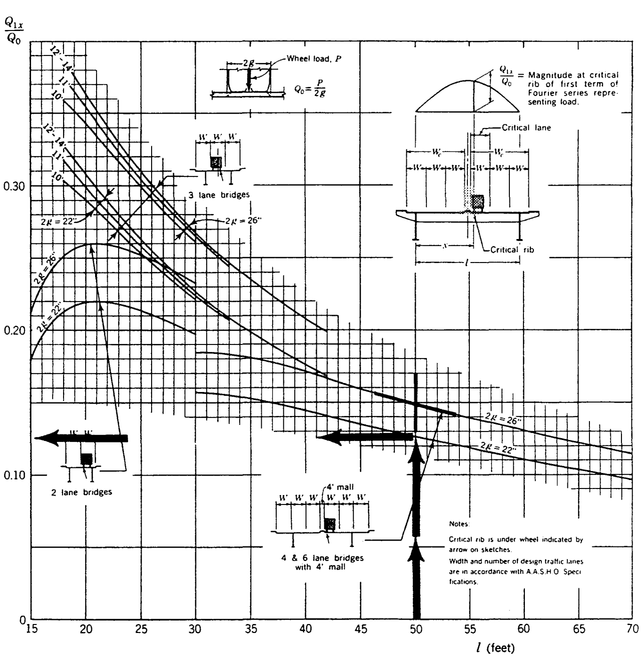


Chart 28. Computation of additional moment in ribs due to floor beam flexibility. Values of $Q_{1x}Q_0$ at critical rib, lane over critical rib loaded

Figure 13

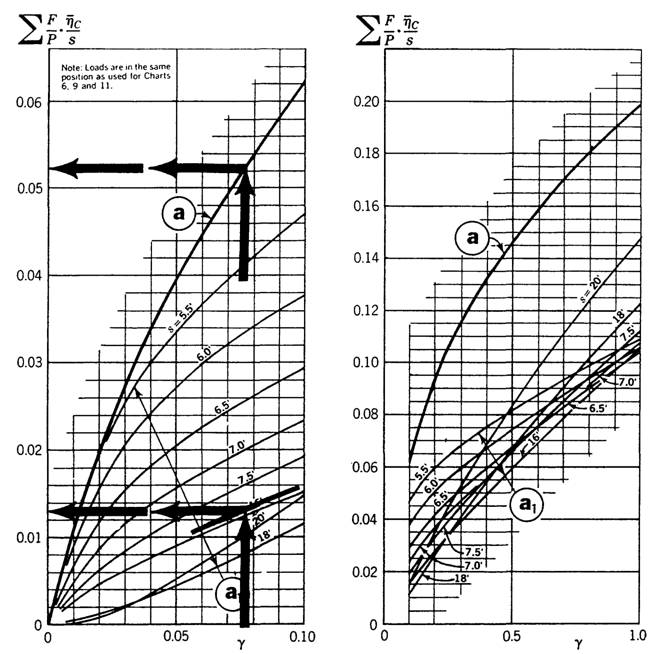


Chart 20. Computation of additional moment at midspan of ribs due to floor beam flexibility. Values of $\Sigma(F/P)(\bar{\eta}_C/s)$, loading a and a_1

Figure 14

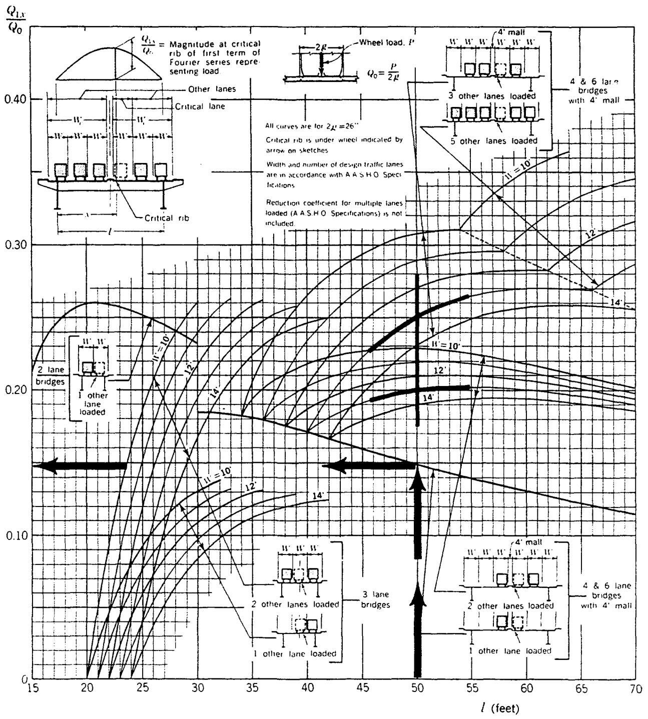


Chart 29. Computation of additional moment in ribs due to floor beam flexibility. Values of $Q_{1x}Q_0$ at critical rib, lane over critical rib loaded

Figure 15

Combining these separate factors the value for ΔM_R equals 0.709(180)(24)(0.126)(0.0526) = 20.3 k-in./rib for loading condition **a**.

The value for loading condition $\mathbf{a_1}$ is similarly determined from the same charts as follows:

$$M_R = 0.709(180)(24)(0.126)(0.0132) = 5.1 \text{ k-in./rib}$$

Since deflections of floor beams have a marked effect on the moments in the ribs, the effect of loads in other lanes must also be taken into account. These have not been previously considered. The same equation used for the effects of floor beam flexibility due to the load applied in the lane considered is applicable. The first three terms can be determined as previously with the modification that 32 kip axle loads should be considered. The value for Q_{1x}/Q_o, 0.149, may be read from Fig. 15 (Chart 29) opposite the intercept of 50 ft span and the curve for one other lane loaded. Notice that higher values could be selected for the case of two other lanes loaded and three other lanes loaded. This would not produce the maximum net moment, however, since AASHO Specifications permit a 10 percent decrease in moments for the case of three lanes loaded and a reduction of 25 percent for the case of four lanes loaded. Such a reduction of the final moment for the case being considered would result in a smaller moment in the end. Fig. 16 (Chart 23) gives a value for the term $\sum (F_m/P)(\bar{\eta}_{im}/s)$ which is equal to 0.0770.

Combining these terms, the increase in moment at mid-span per rib due to loading in other lanes may be determined by

$$\Delta M_R = 0.800(180)(24)(0.149)(0.0770) = 39.6 \text{ k-in./rib}$$

Summation of the dead load moments, the live load moment for ribs on rigid supports, and the increase or decrease in moments due to the effect of floor beam flexibility produces a net moment which may be employed in the subsequent calculations of combined stresses in the rib and deck plate.

MOMENTS IN FLOOR BEAMS

A similar approach is appropriate for the calculation of moments in floor beams. Calculate the dead load moments by the conventional formulas for simple beams, $wl^2/8$. Determine the dead load moments induced by the cantilevered sidewalk by usual formulas. Erection stresses, which would vary depending upon the erection procedure and may or may not be involved in the final maximum moments, are calculated by usual formulas.

Live load moments, disregarding the effect of the flexibility of the floor beam, are obtained by charts which provide the factors of the expression

$$M_F = \frac{M}{F_o l}\left(\frac{F_o}{P}\right) Pl$$

The first term may be evaluated by use of Fig. 17 (Chart 16). For the case of two lanes loaded, its value would be 0.360, and for three lanes loaded 0.378. Fig. 18 (Chart 15), gives the value of F_o/P, a factor which takes account of the critical placement of the wheels and has a value of

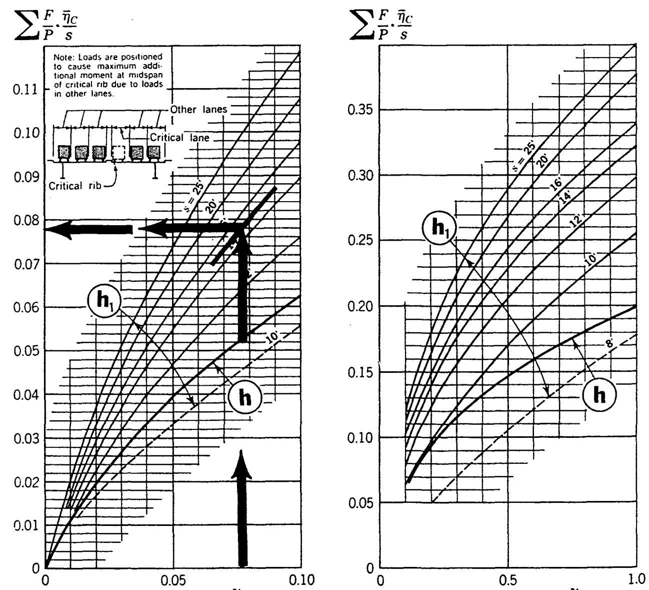


Chart 23. Computation of additional moment at midspan of ribs due to floor beam flexibility. Values of $\Sigma(F/P)(\bar{\eta}_C/s)$, other than critical lanes loaded

Figure 16

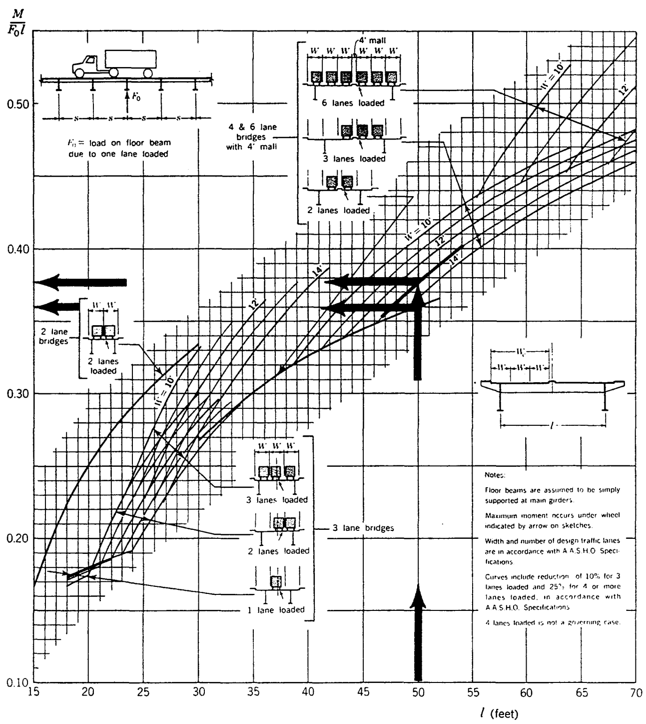


Chart 16. Maximum live load moment in a rigid floor beam

Figure 17

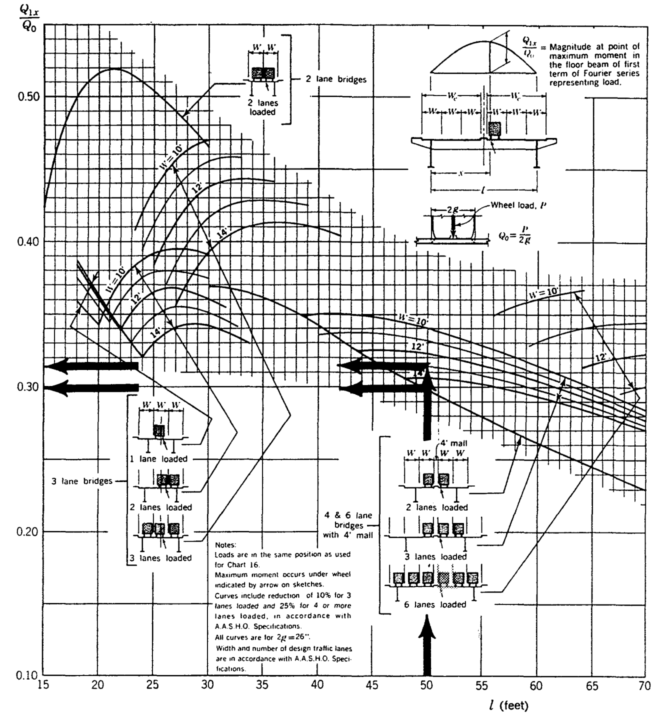


Chart 32. Computation of moment relief in a floor beam due to floor beam flexibility. Values of $Q_{1x}Q_0$ at critical point of a floor beam

Figure 19

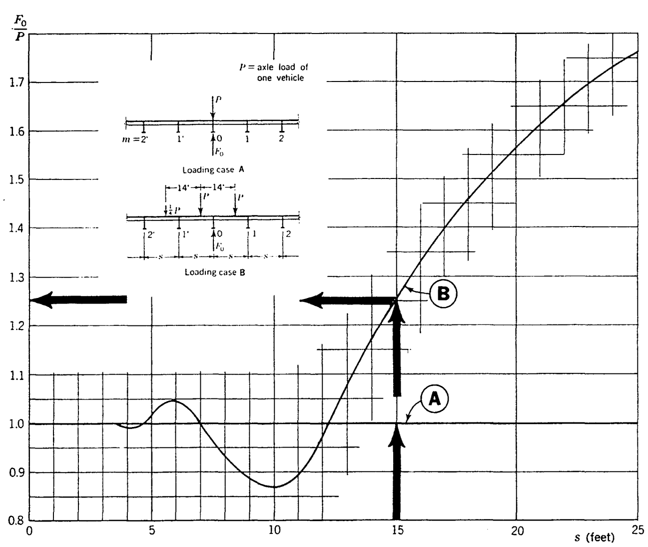


Chart 15. Maximum load, F_0, on a rigid floor beam due to one AASHO vehicle

Figure 18

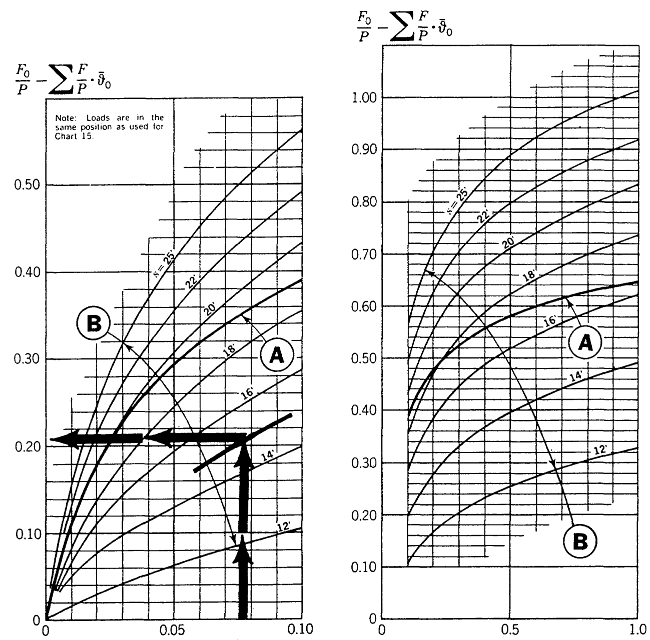


Chart 31. Computation of moment relief in a floor beam due to floor beam flexibility. Values of $(F_0/P) - \Sigma(F/P)\bar{\vartheta}_0$ for $s = 12'$ to $25'$

Figure 20

1.25 for the problem considered. P equals the axle load, 32 kips, times the impact factor, 1.286. Dimension l equals the span length of floor beam, in feet. The product of the several factors for the floor being considered is as follows:

$M_F = 0.360(1.25)(41.4)(50) = 925$ k-ft (2 lanes loaded)

$M_F = 0.378(1.25)(41.4)(50) = 973$ k-ft (3 lanes loaded)

The effect of floor beam flexibility which results in a reduction of the reactions of the deck upon the floor beam and thus a reduction in floor beam moments may be calculated by the following formula:

$$\Delta M_F = Q_o \left(\frac{l}{\pi}\right)^2 \frac{Q_{1x}}{Q_o} \left[\frac{F_o}{P} - \sum \frac{F_m}{P} \bar{\vartheta}_{om}\right] \quad \text{Eq. (5.14)*}$$

The first two terms may be calculated as before. The Q_{1x}/Q_o term may be read from Fig. 19 (Chart 32) as 0.298 for 2 lanes loaded, and 0.315 for three lanes loaded. The final term in the brackets may be determined by the use of Fig. 20 (Chart 31) as 0.31. The product of these terms then will be

$$\Delta M_F = 9.5 \left(\frac{50}{\pi}\right)^2 (0.298)(0.21) =$$

—151 k-ft (2 lanes loaded)

* *Equation number as given in Reference 1.*

$$\Delta M_F = 9.5 \left(\frac{50}{\pi}\right)^2 (0.315)(0.21) =$$

—159 k-ft (3 lanes loaded)

In the case of charts for floor beam moments with more than two lanes loaded the reductions allowed under AASHO Specifications have been taken into account. Therefore, no further reduction is warranted.

The algebraic sum of the separate moments determined as above are the moments which should be used in calculating stresses in the floor beams and deck plate acting as the top flange.

Finally, combining the deck plate stresses at the critical points will provide a close approximation of the maximum stresses in the deck plate.

CONCLUSION

Design of steel plate deck bridges, based upon the methods presented in the AISC orthotropic plate design manual, provide solutions that check within 5 percent of the stresses observed in actual tests. Design by these methods is vastly more simple than design by purely theoretical methods. It is hoped that through experience in the United States, greater knowledge of the art will develop, and that subsequent editions of the manual will reflect further advances in knowledge of the design and application of the orthotropic plate principles.