# Lateral Wind Bracing Requirements for Steel Composite Bridges

C. P. HEINS, C. K. HOU, AND H. KATO

The design of moderate multi-span composite bridges requires utilization of cross frame diaphragms between girders and bottom lateral wind bracing.

The AASHTO code[1] specifies that cross frames or diaphragms shall be provided at each end of the bridge and intermediate frames spaced at intervals not to exceed 25 ft. Previously, AASHTO specifications required that bottom lateral bracing should be provided, located near the bottom of the flange, on spans of 125 ft or longer, if the system has a concrete floor or a floor of equal rigidity.

It is the intention of this paper to present the results of a recent study,[2] which evaluated the effectiveness of bracing in redistributing wind load and formed the basis for the lateral bracing provisions of the 1982 AASHTO Interim Specifications. This effect has been determined by examining the response of single-span, multi-girder, composite bridges, when subjected to wind load. The bridges examined had span lengths of 80 ft, 120 ft, 180 ft, and 300 ft, with girder spacings of 6 ft, 10 ft, and 14 ft.

The resulting response of these bridges indicates that the bottom lateral bracing has minimal effects on the redistribution of the wind load. The important parameter in load redistribution is the diaphragm spacing and span length.

The utilization of cross diaphragms, in reducing the flange stresses due to wind loading, are given in a series of design equations.

## THEORY

**Bridge Model**—The analysis of any bridge structure requires proper idealization prior to performing the analysis. Since the advent of matrix formulations and the computer, the engineer can idealize the structure as a beam, grid or space frame.

*C. P. Heins is Professor, Institute for Physical Science and Technology and Civil Engineering Dept., University of Maryland, College Park, Md.*

*C. K. Hou is a Graduate Research Assistant, Civil Engineering Dept., University of Maryland, College Park, Md.*

*H. Kato is a Research Assistant, Civil Engineering Dept., University of Maryland, College Park, Md.*

In this study, the interaction of bottom lateral bracing, cross diaphragms, and longitudinal girders, when subjected to wind loading along the outside girder, was to be determined. The idealization of such a system required use of a space frame[6] formulation, a three-dimensional (3-D) modeling. Such idealization permitted incorporation of bottom lateral bracing restraint along the bottom flange which could not be considered if a plane grid formulation was utilized. The 3-D model will result in determination of three moments and three vector forces at each end of a given member, which can then be resolved into stress resultants.

The procedure that was used to model the actual bridge into the 3-D system, was to idealize the top and bottom flanges of each longitudinal girder as beam members. The beams were then connected rigidly together by vertical and diagonal web elements.

A typical longitudinal composite girder, removed from the bridge system, is shown in Fig. 1. This basic girder was then idealized as a space frame, shown in Fig. 2. Selection of the proper dimensional stiffness of the flanges and web elements of this 3-D model was then required. The properties of the bottom flange of the model were assumed to be identical to those of the prototype. The properties of the top flange consisted of the combined effect of the steel flange and effective transformed concrete deck. The web elements (diagonals and verticals) were determined by assuming various sizes and then examining the response of the resulting 3-D model (Fig. 2) to that of the composite girder (Fig. 1), when subjected to the same loading. When the response of the 3-D model agreed with the actual girder then the proper web sizes had been obtained. Comparative response between the prototype and 3-D model included comparisons between top and bottom flange stresses, vertical deflection, and end rotation.

**Properties**—The prototype bridges to be examined consisted of multiple girders, in composite action with a concrete deck. The girder properties of these bridges are shown in Table 1 for various span lengths and girder spacings.

Using these properties and performing the modeling transfer just described, has resulted in the girder properties shown in Table 2. A comparison between the resulting

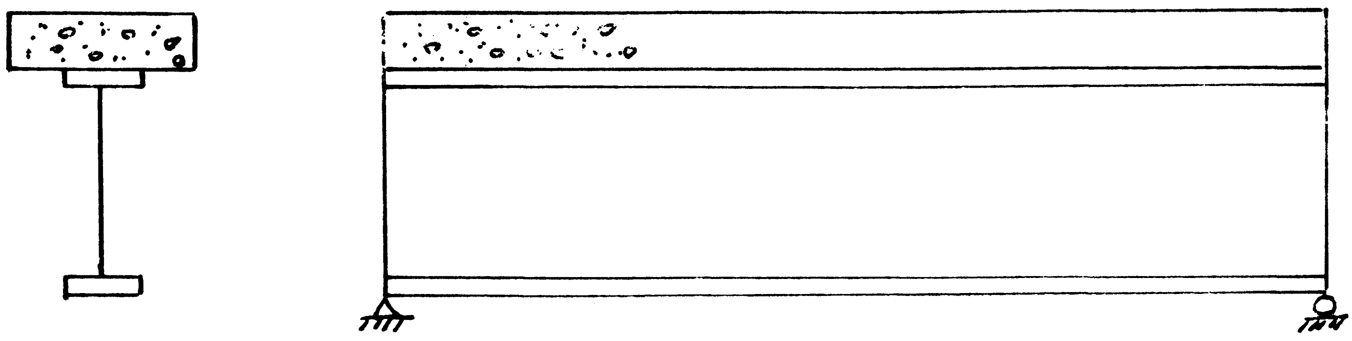

*Fig. 1. Actual single girder*

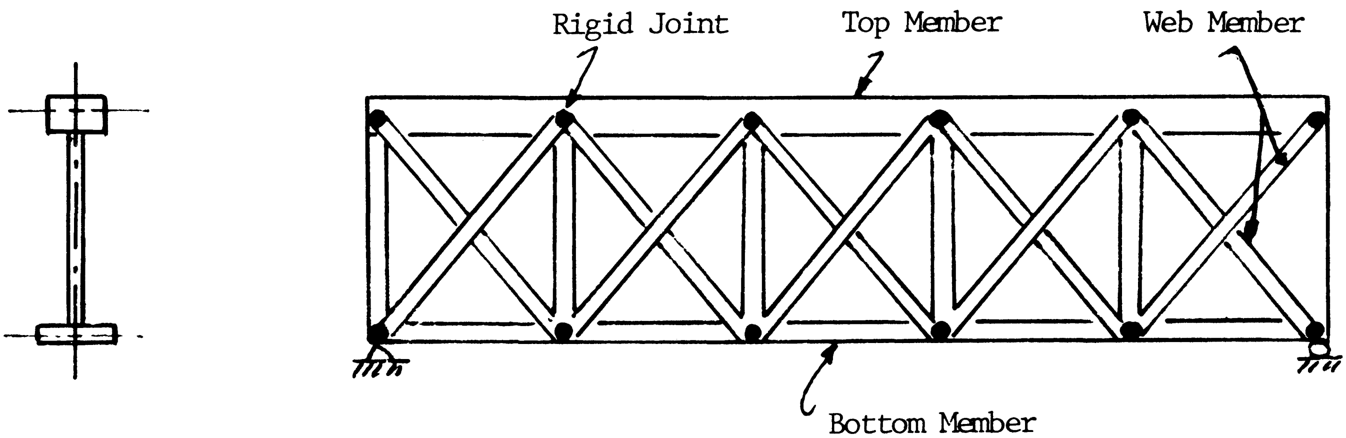


*Fig. 2. Frame model*

response of the 3-D model and the prototype are shown in Table 3.

**3-D Model**—The resulting three-dimensional model, incorporating each idealized longitudinal girder cross diaphragms, deck and bottom lateral bracing, is shown in Fig. 3. This complex system was then used to examine the wind load effect on the braced system.

### TRUCK LOAD RESPONSE

**Analytical Live Load Distribution**—Prior to studying the response of the various bridges, when subjected to wind loading, a comparison between typical prototype bridge live load distributions and present AASHTO[1] code criteria was made. Using the modeled space frame bridge, for spans of 80 ft, 120 ft, and 180 ft and girder spacings of 6 ft, 10 ft, and 14 ft, a series of HS20-44 vehicles were placed on the structure. The resulting live-load distribution factor was obtained[2] for each structure and resulted in an average $K = 6.7$. This factor is not unreasonable as the AASHTO value of 5.5 represents an average as determined from a previous study[4] where it was found that the $K$ factor varied from 4.7 to 7.5.

**Table 1. Prototype Bridge Girder Geometry**

| L | $S_g$ | WEB (inxin) | BOTTOM FLANGE (inxin) | TOP FLANGE (inxin) |
|---|---|---|---|---|
| 80' | 6' | 60x7/16 | 14x5/8 | 12x9/16 |
| | 10' | 60x9/16 | 20x1 | 12x9/16 |
| | 14' | 66x5/8 | 24x19/16 | 15x11/16 |
| 120' | 6' | 66x1/2 | 22x1 | 15x11/16 |
| | 10' | 75x1/2 | 27x31/16 | 21x15/16 |
| | 14' | 84x9/16 | 33x3/2 | 24x19/16 |
| 180' | 6' | 90x5/8 | 28x5/4 | 23x1 |
| | 10' | 102x5/8 | 36x13/8 | 30x11/8 |
| | 14' | 108x11/16 | 43x2 | 36x27/16 |
| 300' | 6' | 120x3/4 | 47x33/16 | 43x2 |
| | 10' | 144x7/8 | 55x5/2 | 52x37/16 |
| | 14' | 150x1 | 59x27/8 | 59x47/16 |

Table 2. Space-frame Girder Model Geometry

| L(ft) | $S_g$ ft | | A($in^2$) | $K_T$($in^4$) | $I_X$($in^4$) | $I_Y$($in^4$) |
|---|---|---|---|---|---|---|
| 80 | 6 | Bottom | 8.75 | 1.14 | 0.285 | 143.0 |
| | | Top | 62.75 | 1030.00 | 314.0 | 24300 |
| | | Web(V) | 150 | 1.0 | 1.0 | 1.0 |
| | | Web(D) | 150 | 1.0 | 1.0 | 1.0 |
| | 10 | Bottom | 20 | 6.67 | 1.67 | 667 |
| | | Top | 115 | 3280 | 820 | 105000 |
| | | Web(V) | 200 | 1.0 | 1.0 | 1.0 |
| | | Web(D) | 200 | 1.0 | 1.0 | 1.0 |
| | 14 | Bottom | 28.5 | 13.4 | 3.35 | 1368 |
| | | Top | 171 | 7320 | 1957 | 234000 |
| | | Web(V) | 200 | 1.0 | 1.0 | 1.0 |
| | | Web(D) | 250 | 1.0 | 1.0 | 1.0 |
| 120 | 6 | Bottom | 22 | 7.32 | 1.83 | 887 |
| | | Top | 66 | 1030 | 283 | 24200 |
| | | Web(V) | 250 | 1.0 | 1.0 | 1.0 |
| | | Web(D) | 220 | 1.0 | 1.0 | 1.0 |
| | 10 | Bottom | 52.3 | 65.6 | 16.4 | 3178 |
| | | Top | 128.0 | 3280 | 1140 | 105000 |
| | | Web(V) | 400 | 1.0 | 1.0 | 1.0 |
| | | Web(D) | 400 | 1.0 | 1.0 | 1.0 |
| | 14 | Bottom | 49.5 | 37.2 | 9.28 | 4492 |
| | | Top | 190 | 7321 | 2525 | 234000 |
| | | Web(V) | 400.0 | 1.0 | 1.0 | 1.0 |
| | | Web(D) | 400.0 | 1.0 | 1.0 | 1.0 |
| 180 | 6 | Bottom | 35 | 18.4 | 4.6 | 2287 |
| | | Top | 79 | 1030 | 489 | 24200 |
| | | Web(V) | 750 | 1.0 | 1.0 | 1.0 |
| | | Web(D) | 750 | 1.0 | 1.0 | 1.0 |
| | 10 | Bottom | 58.5 | 51.6 | 12.9 | 6318 |
| | | Top | 149 | 3280 | 1521 | 105000 |
| | | Web(V) | 450 | 1.0 | 1.0 | 1.0 |
| | | Web(D) | 450 | 1.0 | 1.0 | 1.0 |
| | 14 | Bottom | 86 | 114.4 | 28.6 | 13251 |
| | | Top | 222 | 7321 | 3399 | 234000 |
| | | Web(V) | 500 | 1.0 | 1.0 | 1.0 |
| | | Web(D) | 500 | 1.0 | 1.0 | 1.0 |
| 300 | 6 | Bottom | 96.94 | 137.45 | 34.36 | 17845 |
| | | Top | 142 | 1144 | 944.1 | 37443 |
| | | Web(V) | 1750 | 1.0 | 1.0 | 1.0 |
| | | Web(D) | 1750 | 1.0 | 1.0 | 1.0 |
| | 10 | Bottom | 137.5 | 286.5 | 71.6 | 34661 |
| | | Top | 228.3 | 3495 | 2603 | 132072 |
| | | Web(V) | 3000 | 1.0 | 1.0 | 1.0 |
| | | Web(D) | 3000 | 1.0 | 1.0 | 1.0 |
| | 14 | Bottom | 199 | 756 | 189 | 57763 |
| | | Top | 335 | 7819 | 5810 | 284531 |
| | | Web(V) | 3000 | 1.0 | 1.0 | 1.0 |
| | | Web(D) | 3000 | 1.0 | 1.0 | 1.0 |

**Experimental Live Load Distribution**—In order to further demonstrate the reliability of the space frame model, the response of an actual bridge which was field-tested[5] has been examined. The bridge, located on the Patuxent River in Maryland, was field-tested in 1968 and is shown in Fig. 4. The bridge was subjected to a simulated HS20 truck which was positioned on the bridge also shown in Fig. 4. The resulting experimental bottom flange strains and center line girder deflections are shown in Fig. 5.

The bridge was then modeled as a 3-D frame using the techniques described previously. The model was then subjected to the field test loading and positioned on the model per Fig. 4. The resulting stresses and vertical deformations were then obtained as shown in Fig. 5. As seen from these comparisons, reasonable agreement occurs between theory and tests.

Table 3. Response of Single-Girder Prototype and Model

| L (ft) | $S_g$ (ft) | 6 | | 10 | | 14 | |
|---|---|---|---|---|---|---|---|
| | | PROTOTYPE | MODEL | PROTOTYPE | MODEL | PROTOTYPE | MODEL |
| 80 | T. STR. (ksi) | 1.25 | 1.48 | 0.64 | 0.72 | 0.39 | 0.41 |
| | B. STR. (ksi) | 4.08 | 3.74 | 2.19 | 2.00 | 1.44 | 1.42 |
| | DEFL. (in) | 0.21 | 0.22 | 0.11 | 0.11 | 0.064 | 0.069 |
| | ROTA. $x10^{-4}$ | 6.59 | 6.96 | 3.44 | 3.56 | 2.00 | .216 |
| 102 | T.STR. (ksi) | 1.48 | 1.67 | 0.66 | 0.68 | 0.41 | 0.41 |
| | B. STR. (ksi) | 3.03 | 3.00 | 1.32 | 1.25 | 1.14 | 1.16 |
| | DEFL. (in) | 0.37 | 0.40 | 0.14 | 0.14 | 0.097 | 0.10 |
| | ROTA. $x10^{-4}$ | 7.60 | 8.34 | 2.87 | 3.00 | 2.02 | 2.17 |
| 180 | T. STR. (ksi) | 1.33 | 1.60 | 0.64 | 0.68 | 0.41 | 0.41 |
| | B. STR. (ksi) | 2.16 | 2.01 | 1.22 | 1.21 | 0.82 | 0.81 |
| | DEFL. (in) | 0.47 | 0.52 | 0.22 | 0.24 | 0.14 | 0.15 |
| | ROTA. $x10^{-4}$ | 6.56 | 7.21 | 3.06 | 3.32 | 1.88 | 2.02 |
| 300 | T. STR. (ksi) | 2.36 | 2.71 | 1.24 | 1.43 | 0.81 | 0.91 |
| | B. STR. (ksi) | 3.20 | 3.17 | 1.86 | 1.84 | 1.26 | 1.26 |
| | DEFL. (in) | 1.75 | 1.92 | 0.82 | 0.90 | 0.53 | 0.57 |
| | ROTA. $x10^{-4}$ | 1.53 | 1.69 | 0.73 | 0.80 | 0.47 | 0.51 |

This basic data thus provides some assurance as to the reliability of the modeling technique and it is now possible to examine analytically the various bridges, when subjected to lateral wind loads.

## WIND LOAD STUDY

**Parameters**—As described in Table 1, the composite bridges to be examined have span lengths of 80 ft, 120 ft, 180 ft and 300 ft, with girder spacings of 6 ft, 10 ft, and 14 ft. Attached to these bridges will be two types of bracing configurations as shown in Fig. 6; *x* type and *v* type. Such configurations were used for both lateral wind bracing and cross diaphragms. The cross diaphragms were spaced at 15 ft, 20 ft and 25 ft respectively. Four to six girder systems were considered. The average area of all bracing elements was assumed equal to 4 $in^2$.

The bridge configuration shown in Fig. 6 is a tangent system with right angle supports. In addition to this system, skew supports with diaphragms parallel to the skew supports or normal to the girders have been examined.[3] The angle of skew varied from 0° to a maximum of 30°.

**Loading**—According to the AASHTO code,[1] a uniform loading applied at right angles to the longitudinal axis of the structure, of intensity equal to 50 psf, shall be applied. This load however shall not be less than 300 plf.

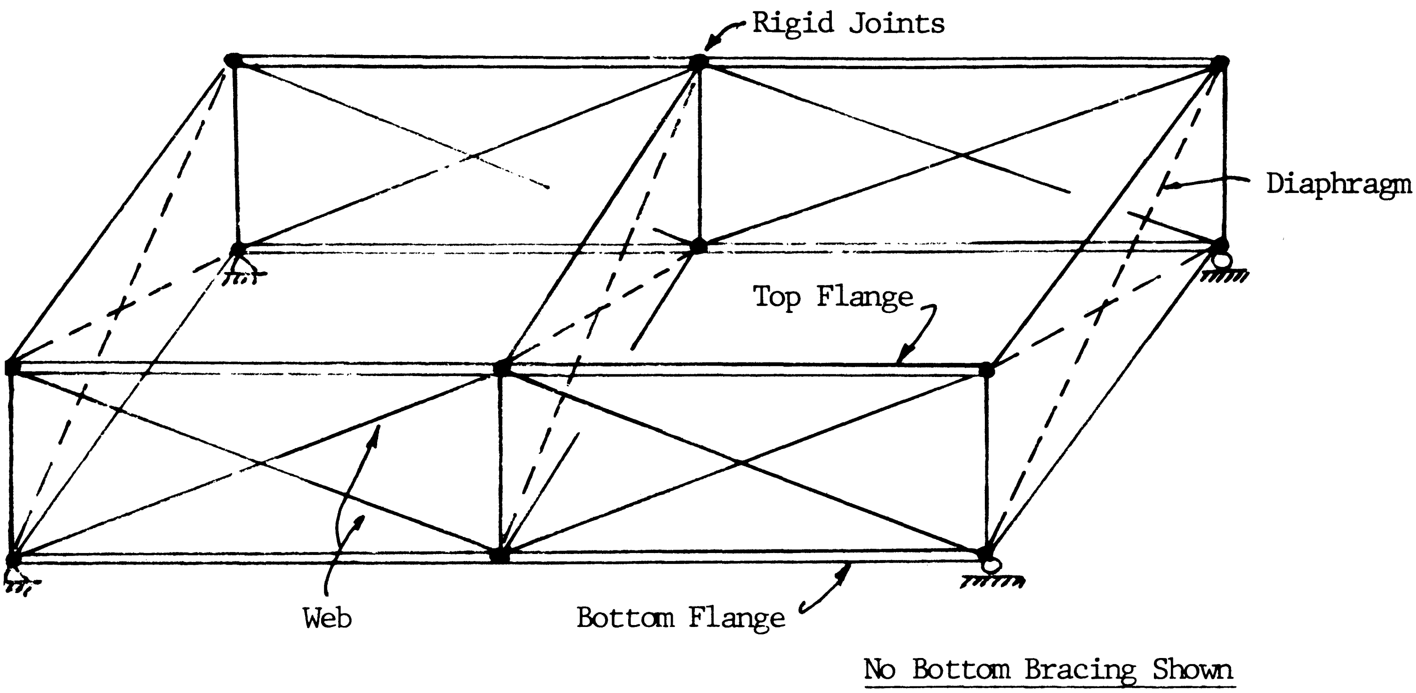


*Fig. 3. Idealized bridge system as 3-D frame*

This loading was then applied to all the modeled bridges. The bridge flange stresses were then evaluated when the bridge had wind bracing and did not have wind bracing.

**Results**—For the various span lengths, the maximum bottom flange bending stresses were determined for the braced and unbraced sections. These stresses were then plotted[2] as a function of diaphragm spacing and girder spacing, as shown in Fig. 7 for a span length of $L = 300$ ft. These results show the importance of the braced to unbraced condition, but more importantly the magnitude of the stresses. Similar results were obtained for the other span lengths.[2]

**Equivalent System**—The behavior of the bottom flange of plate girder system, due to the existence of diaphragms, can be considered as equivalent to a continuous beam which is subjected to uniform loading $w$, as shown in Fig. 8. The properties of the continuous beam are equal to the bottom flange of the girder, and the distance $S_d$ is the cross diaphragm spacing and $w$ is the wind loading.

The stress in this continuous beam can be used as the basic parameter in design, and can be determined from the following equation:

$$f_{\text{con.bm}} = M_{\text{con.bm}} \Big/ \left(\frac{t_f b_f^2}{6}\right) \qquad (1)$$

where:

$t_f$ = thickness of the bottom flange of the girder

$b_f$ = width of the bottom flange of the girder

$M_{\text{con.bm}}$ = maximum moment of the equivalent continuous beam, positive or negative

Using this known stress and the stress evaluated in the bottom flange as obtained from the 3-D analysis and designated as ($f_{\text{syst}}$), a ratio;

$$R = f_{\text{syst}}/f_{\text{con.bm}} \qquad (2)$$

can be formulated.[2] Using this ratio and the diaphragms spacing $S_d$, girder spacing $S_g$ and span length $L$, a relationship between these parameters can be determined.

A comparison for just the 180-ft span bridge is shown in Fig. 9. This plot shows the variation of stress ratio $R$ to diaphragm spacing for the various girder spacings, for the braced and unbraced systems. The maximum positive stress values were used in evaluating the stress ratio $R$, as this gave the maximum effect. Examination of Fig. 9 shows that girder spacing has some influence on the stress ratio $R$, but is minor. Therefore the final plots of the factor $R$ vs. $S_d$, for the unbraced and braced systems as shown in Figs. 10 and 11, respectively, neglect $S_g$ and are plotted for maximum effects.

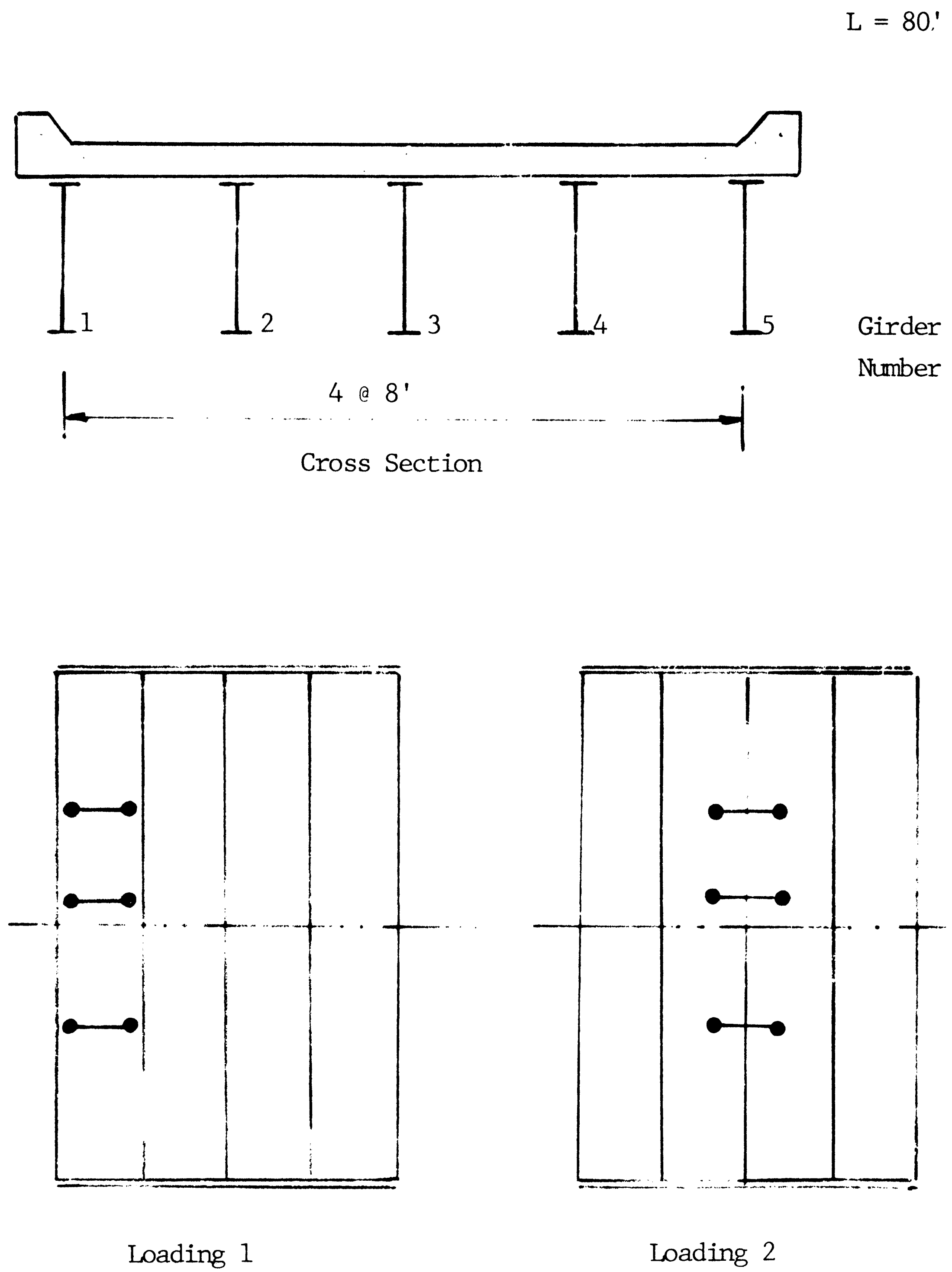


*Fig. 4. Bridge test details*

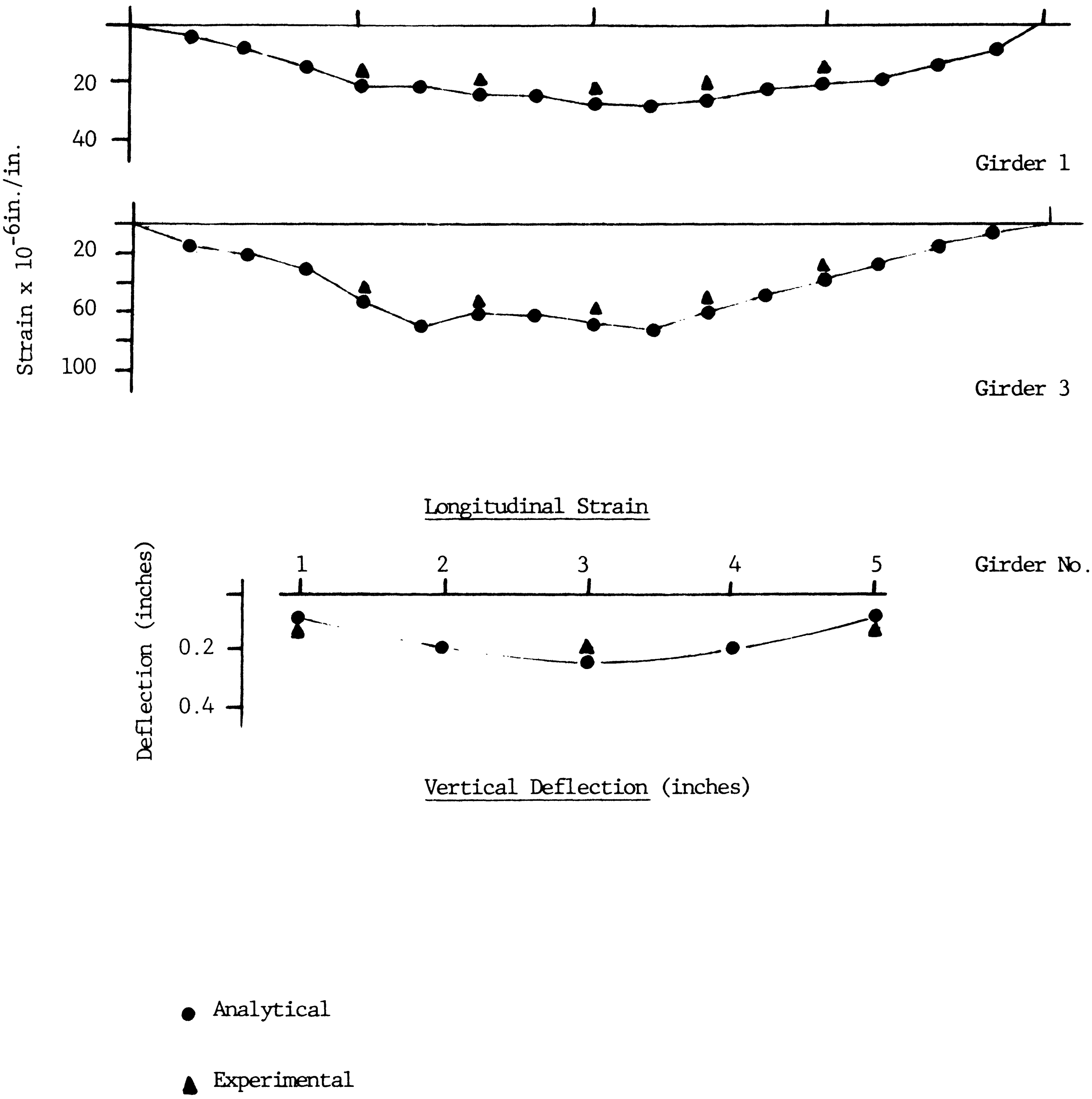


*Fig. 5. Test vs. theory*

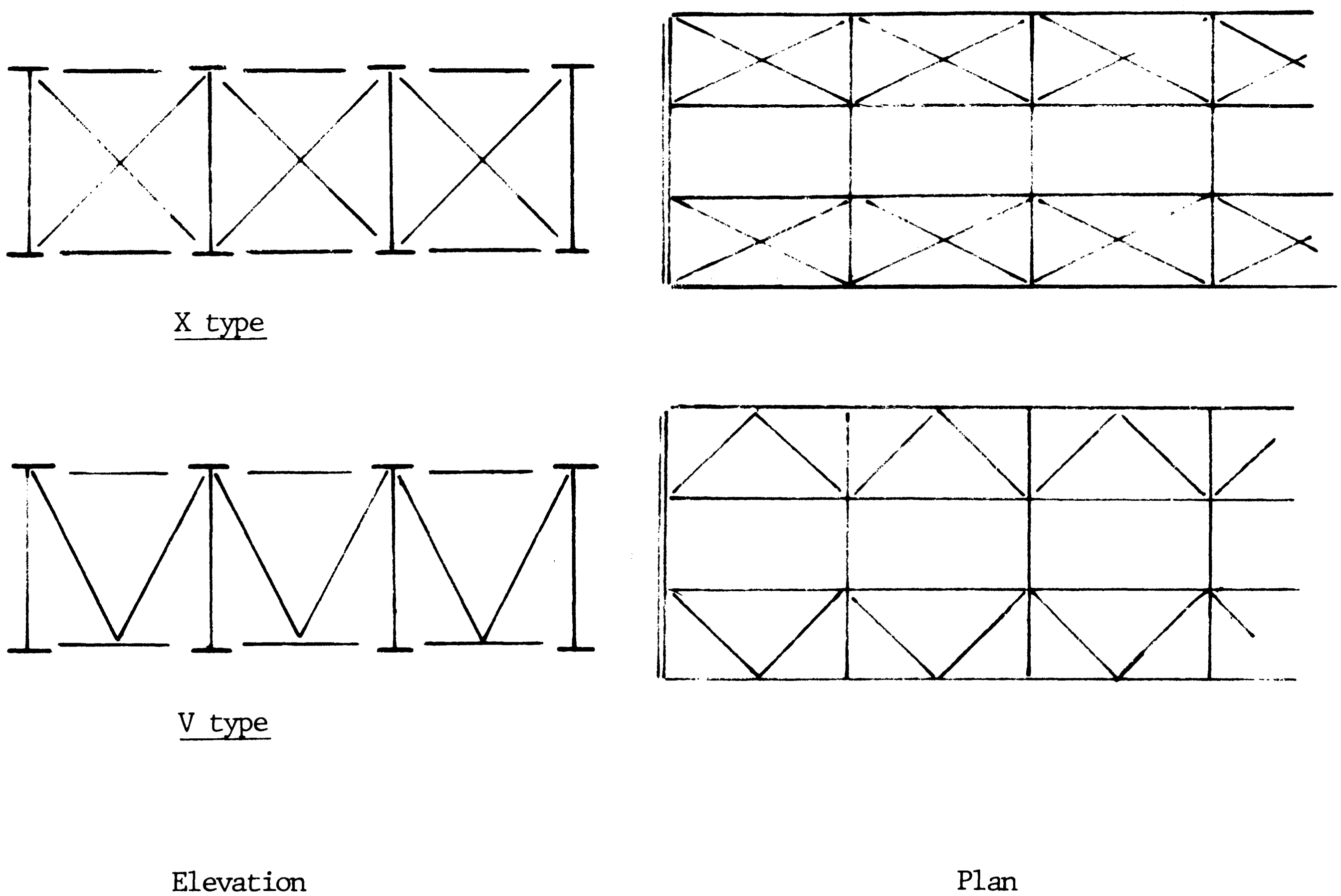


*Fig. 6. Bracing types*

### DESIGN CRITERIA

Using the data shown in Figs. 10 and 11, the stress ratio $R$ has been determined as a function of span length and diaphragm spacing.[2] The general response given in Figs. 10 and 11 is for tangent bridges. A comparison between these values and those for skewed bridges[3] shows minimal differences. Thus the proposed design equations will be applicable for both tangent and right angle supported bridges.

The resulting design equations also shown in Figs. 10 and 11 have been incorporated into Article 1.7.17, Diaphragms, Cross Frames and Lateral Bracing, of the AASHTO code and are to be used as follows:

**(A) General.** Rolled beam and plate girder spans shall be provided with cross frames or diaphragms at each support and with intermediate cross frames or diaphragms placed in all bays and spaced at intervals not to exceed 25 ft. Diaphragms shall be at least $\frac{1}{2}$ and preferably $\frac{3}{4}$ the girder depth. Cross frames shall be as deep as practicable. Intermediate cross frames shall preferably be of the cross type or vee type. End cross frames or diaphragms shall be proportioned to adequately transmit all the lateral forces to the bearings. Intermediate cross frames shall be normal to the main girders when the supports are skewed more than 20°. Cross frames on horizontally curved steel girder bridges shall be designed as main members, with adequate provisions for transfer of lateral forces from the girder flanges.

The need for lateral bracing shall be investigated. Top flanges attached to concrete decks or other decks of comparable rigidity in a manner which provides lateral restraint will not require top lateral bracing. Bottom lateral wind bracing may or may not be provided at the discretion of the engineer, providing the stresses in the bottom flanges due to wind loading are accounted for as specified in Article 1.1.17(B).

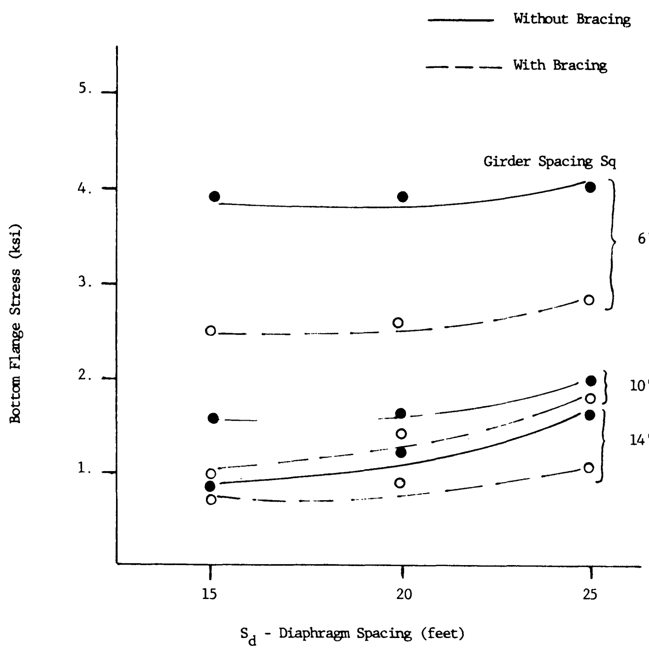


*Fig. 7. Maximum bottom flange stresses—L = 300′*

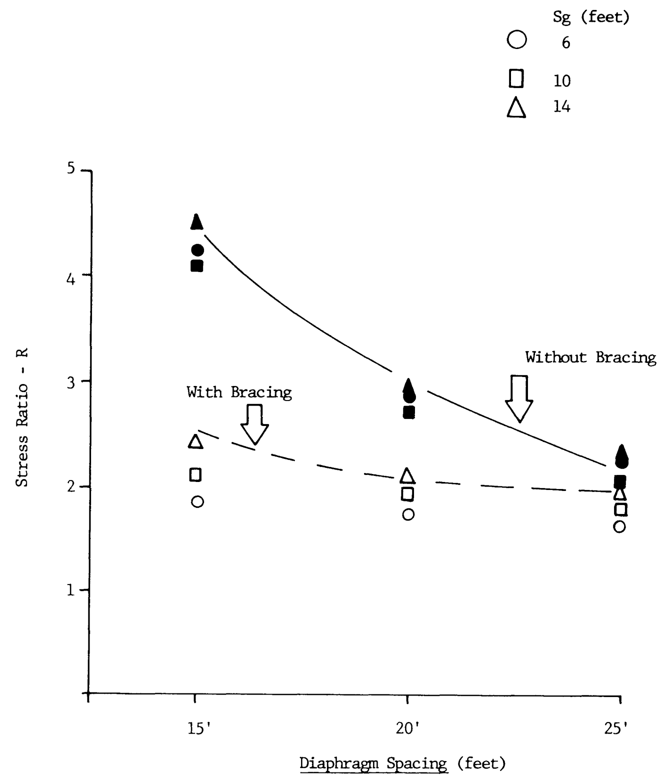


*Fig. 9. Equivalent stress ratio vs. $S_d$ for $L = 180$*

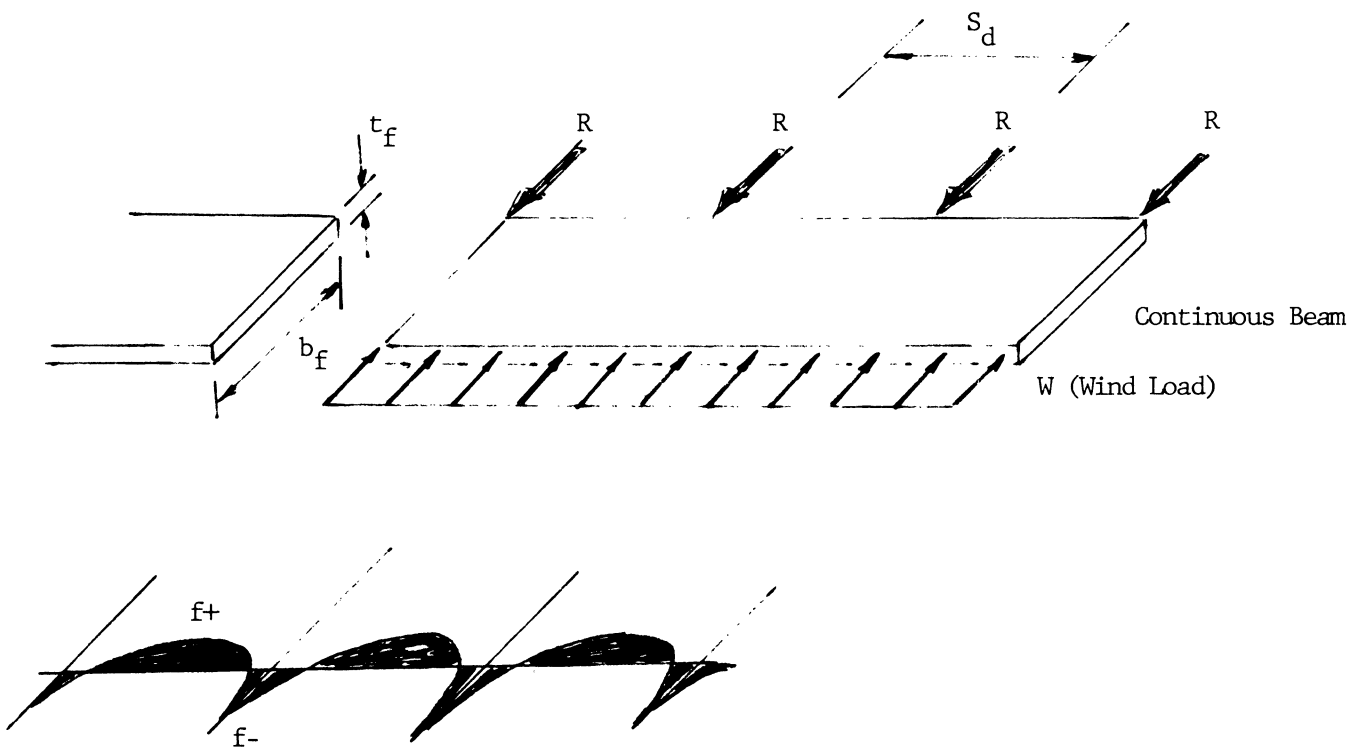


*Fig. 8. Equivalent system*

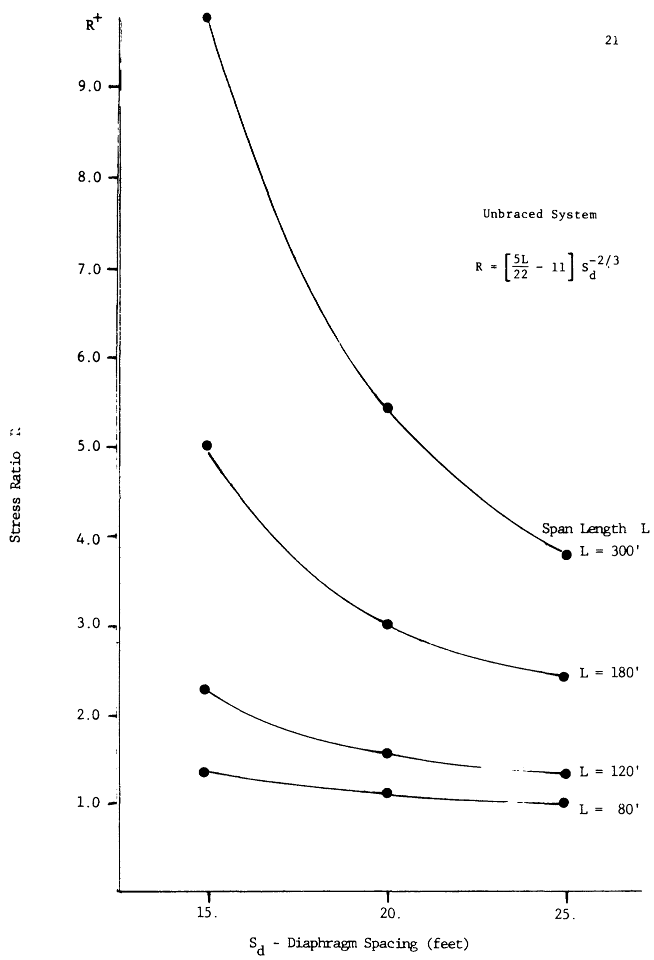


*Fig. 10. Equivalent stress ratio—unbraced system*

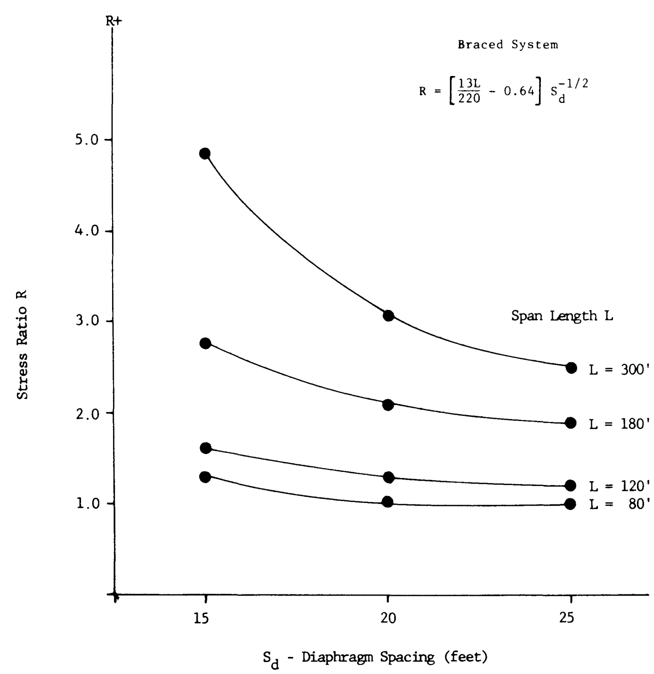


*Fig. 11. Equivalent stress ratio—braced system*

When required, lateral bracing shall be placed in exterior bays between diaphragms or cross frames and in or near the plane of the flange being graced.

Horizontal wind forces in accordance with Article 1.2.14 shall be applied to the area of superstructure exposed in elevation. Half of the force shall be applied in the plane of each flange. The allowable stress shall be in accordance with Article 1.2.22.

**(B) Stresses Due to Wind Loading When Top Flanges are Continuously Supported.**

(*1*) *Flanges.* The maximum induced stress in the bottom flange ($F$) can be computed from the following:

$$F = RF_{cb}$$

where:

$$R = \left[\frac{5L}{22} - 11\right] S_d^{-2/3} \quad \text{when no bottom lateral bracing is provided}$$

$$R = \left[\frac{13L}{220} - 0.64\right] S_d^{-1/2} \quad \text{when bottom lateral bracing is provided}$$

$$F_{cb} = \frac{M_{cb}}{t_f b_f / 6}$$

$$M_{cb} = 0.08 W S_d^2$$

$W$ = Wind loading along the exterior flange (psf)

$S_d$ = Diaphragm spacing (ft)

$L$ = Span length (ft)

(*2*) *Diaphragms.* The maximum horizontal force ($F_d$) in the transverse diaphragms and cross frames is obtained from the following:

With or without bracing: $F_D = 1.14 W S_d$

## CONCLUSIONS

A series of composite I-girder bridges have been examined when subjected to horizontal wind loading. These bridges, which vary in lengths from 80 ft to 300 ft, have transverse diaphragms but may or may not have bottom lateral bracing. A comparison of the resulting bottom flange stresses to various parameters has permitted the development of a simple design equation to determine these wind induced flange stresses.

In general it is noted that the induced flange stresses are a maximum of 3–4 ksi, without wind bracing. Thus the expense of using such bracing may not be warranted.

## NOTATION

$R = f_{sys}/f_{con.bm}$ = stress ratio

$f_{sys}$ = maximum induced bottom flange stress in the 3-D system

$f_{con.bm}$ = maximum induced bottom flange stress in the continuous beam flange

$L$ = span length
$S_d$ = diaphragm spacing
$M_{cb}$ = moment in the continuous beam
$W$ = lateral wind load
$F_D$ = maximum force in diaphragms
$t_f$ = flange thickness
$b_f$ = flange width


## ACKNOWLEDGMENTS

The work covered in this paper was sponsored by the AISI Committee of Structural Steel and Steel Plate Producers. The work was directed by the Engineering Subcommittee of the Structural Steel and Steel Plate Producers. The Task Force of this subcommittee consisted of the following members:

J. A. Gilligan, Project Supervisor
R. S. Fountain, Project Chairman
R. J. Behling
R. C. Cassano
T. M. Dean
E. V. Hourigan
R. P. Knight
J. T. Kratzer
R. W. Lautensleger
D. A. Linger
W. A. Milek, Jr.
C. Pestotnik
F. D. Sears
W. M. Smith